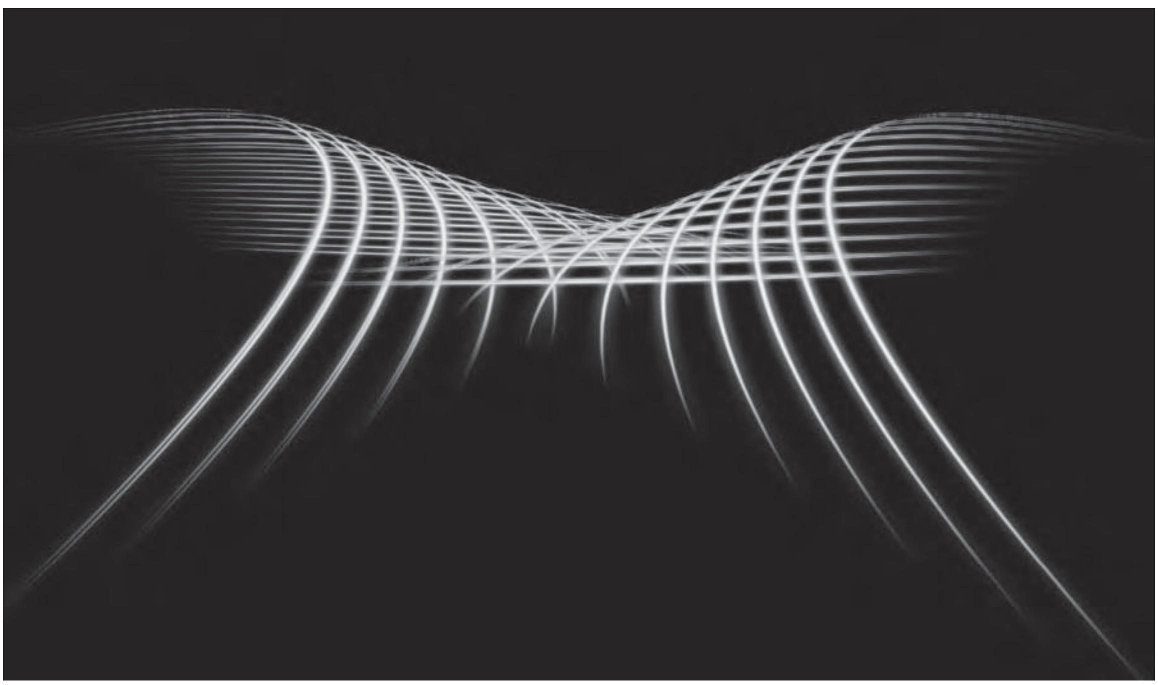
HCM2010

HIGHWAY CAPACITY MANUAL



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LIST OF EXHIBITS

1. INTRODUCTION

The *Highway Capacity Manual 2010* (HCM) is the fifth edition of this fundamental reference document. Like its predecessors, the HCM 2010 has been significantly revised to incorporate the latest research on highway capacity and quality of service. It has also been substantially reorganized. These changes continue the HCM's evolution, keeping the manual in step with its users' needs and present times.

The 1950 HCM (*1*) was the first document to quantify the concept of capacity for transportation facilities and focused almost entirely on that subject. This focus was in response to the rapid expansion of the U.S. roadway system after World War II and the need to determine lane requirements for the Interstate highway system and the roads that provided access to it. The manual was designed to be "a practical guide by which the engineer, having determined the essential facts, can design a new highway or revamp an old one with assurance that the resulting capacity will be as calculated."

The focus on design continued in the 1965 HCM (*2*), but the level-of-service (LOS) concept was also introduced with this edition, along with a chapter on bus transit. The HCM permitted the "determination of the capacity, service volume, or level of service which will be provided by either a new highway design, or an existing highway under specified conditions."

The 1985 HCM (*3*) was another significant step in the evolution of the HCM. It further refined the concept of LOS and incorporated the results of several major research projects performed since the publication of the 1965 HCM. The target audience was broadened through the addition of chapters on pedestrians and bicycles and an expansion of the transit chapter.

A substantial increase in the volume and breadth of material occurred with the publication of the HCM2000 (*4*). The intent of the manual was "to provide a systematic and consistent basis for assessing the capacity and level of service for elements of the surface transportation system and also for systems that involve a series or a combination of individual facilities."

The HCM 2010 has added much new material from research projects completed since the publication of HCM2000 and has been reorganized to make its contents more accessible and understandable. The reorganization is also intended to encourage analysts and decision makers to consider all roadway users, as well as a broader range of performance measures, when they assess transportation facility performance.

Chapter 1, HCM User's Guide, is the starting point for learning how to use this edition of the HCM. This chapter presents the purpose, objectives, intended use, and target users of the HCM 2010; describes the contents of each of the four volumes that make up the HCM; summarizes the major changes that have been made to HCM2000 methodologies; and mentions some of the important companion documents to the HCM. The remainder of Volume 1 presents the fundamental information with which users should be familiar before starting to apply the manual.

2. HCM PURPOSE AND SCOPE

PURPOSE AND OBJECTIVES

Quality of service describes how well a transportation facility or service operates from the traveler's perspective.

The purpose of the HCM is to provide a set of methodologies, and associated application procedures, for evaluating the multimodal performance of highway and street facilities in terms of operational measures and one or more quality-of-service indicators.

The objectives of the HCM are to

1. Define performance measures and describe survey methods for key traffic characteristics,
2. Provide methodologies for estimating and predicting performance measures, and
3. Explain methodologies at a level of detail that allows readers to understand the factors affecting multimodal operation.

Level of service is the A-F stratification of quality of service.

The HCM 2010 presents the best available techniques at the time of publishing for determining capacity and LOS. However, it does not establish a legal standard for highway design or construction.

INTENDED USE

The HCM is intended to be used primarily for the analysis areas listed below, to the extent that they are supported by the individual analysis methodologies.

- *Levels of analysis:* operations, design, preliminary engineering, and planning.
- *Travel modes:* automobile (and other motorized vehicles), pedestrian, and bicycle, plus transit when it is part of a multimodal urban street facility.
- *Spatial coverage:* points, segments, and facilities.
- *Temporal coverage:* undersaturated and oversaturated conditions.

TARGET USERS

The HCM is prepared for use by (*a*) engineers who work in the field of traffic operations or highway geometric design and (*b*) transportation planners who work in the field of transportation system management. To use the manual effectively and to apply its methodologies, some technical background is desirable—typically university-level training or technical work in a public agency or consulting firm.

The HCM is also useful to management personnel, educators, air quality specialists, noise specialists, elected officials, regional land use planners, and interest groups representing special users.

3. STRUCTURE

OVERVIEW

Since 2000, more than $5 million in funded National Cooperative Highway Research Program (NCHRP) research has partially or entirely focused on HCM methodologies. To keep the HCM at a manageable size and yet incorporate the results of this research, the HCM 2010 has been divided into four volumes:

1. Concepts,
2. Uninterrupted Flow,
3. Interrupted Flow, and
4. Applications Guide.

When the HCM2000 (*4*) was being developed, U.S. states were moving toward compliance with federal requirements to use metric units in the design of roadways. As a result, the HCM2000 was published in "U.S. customary" and "metric" versions. Because the federal metrication requirements were later dropped and most states returned to U.S. customary units, this edition only uses U.S. customary units. A metric conversion guide is provided later in this chapter.

The following sections describe the contents of each HCM 2010 volume.

VOLUME 1: CONCEPTS

Volume 1 covers the basic information that an analyst should be familiar with before performing capacity or quality-of-service analyses. Its chapters cover the organization of the HCM; the kinds of applications that can be performed with the HCM; modal characteristics; traffic flow, capacity, and quality-of-service concepts; the range of tools available to perform an analysis; guidance on interpreting and presenting analysis results; and the terms and symbols used in the HCM. Chapter 8, HCM Primer, provides an executive summary of the HCM for decision makers.

Users familiar with the HCM2000 will find that Volume 1 incorporates Part I of the HCM2000; conceptual material from HCM2000 Parts II, IV, and V; and new material developed for the 2010 edition. Volume 1 is provided in three-ring binder and electronic formats, to facilitate the addition of new conceptual material as new research is incorporated into the HCM between major updates.

VOLUME 1: CONCEPTS
1. HCM User's Guide
2. Applications
3. Modal Characteristics
4. Traffic Flow and Capacity Concepts
5. Quality and Level-of-Service Concepts
6. HCM and Alternative Analysis Tools
7. Interpreting HCM and Alternative Tool Results
8. HCM Primer
9. Glossary and Symbols

Chapter 8 serves as an executive summary of the HCM for decision makers.

VOLUME 2: UNINTERRUPTED FLOW

Volume 2 contains the methodological chapters relating to uninterrupted-flow system elements. All of the material necessary in performing an analysis of one of these elements appears here: a description of the process thorough enough to allow an analyst to understand the steps involved (although not necessarily replicate them by hand), the scope and limitations of the methodology, specific default values, LOS thresholds, and guidance on special cases and the use of alternative tools.

The freeway chapters are presented first, arranged from the facility level down to the segment level; the chapters for multilane and two-lane highways

VOLUME 2: UNINTERRUPTED FLOW
10. Freeway Facilities
11. Basic Freeway Segments
12. Freeway Weaving Segments
13. Freeway Merge and Diverge Segments
14. Multilane Highways
15. Two-Lane Highways

Uninterrupted-flow system elements, such as freeways, have no fixed causes of delay or interruption external to the traffic stream.

follow. Users familiar with the HCM2000 will find that Volume 2 incorporates the Part III uninterrupted-flow chapters, along with the material from the corresponding Part II chapters (e.g., specific default values and LOS thresholds) used directly in an analysis. The Interchange Ramp Terminals chapter, which appeared with the uninterrupted-flow chapters in the HCM2000, appears with the interrupted-flow chapters (Volume 3) in the HCM 2010.

Volume 2 is provided in both three-ring-binder and electronic formats to facilitate interim HCM updates as new research is performed.

VOLUME 3: INTERRUPTED FLOW
16. Urban Street Facilities
17. Urban Street Segments
18. Signalized Intersections
19. TWSC Intersections
20. AWSC Intersections
21. Roundabouts
22. Interchange Ramp Terminals
23. Off-Street Pedestrian and Bicycle Facilities

Interrupted-flow system elements, such as urban streets, have traffic control devices such as traffic signals and STOP signs that periodically interrupt the traffic stream.

VOLUME 3: INTERRUPTED FLOW

Volume 3 contains all of the methodological chapters relating to interrupted-flow system elements. Its content is similar to that of the Volume 2 chapters. The facility chapter is presented first, followed by the segment chapter, the point chapters, and a chapter on off-street pedestrian and bicycle facilities.

Volume 3 incorporates the interrupted-flow chapters from the HCM2000's Part III, along with the corresponding detailed Part II material. Where applicable, pedestrian and bicycle material has been integrated throughout the Volume 3 chapters, along with public transit material specific to multimodal analyses. Users are referred to the *Transit Capacity and Quality of Service Manual* (TCQSM) (*5*) for transit-specific analysis procedures. The HCM2000 Unsignalized Intersections chapter has been split into three chapters in the 2010 edition, which individually cover two-way STOP-controlled intersections, all-way STOP-controlled intersections, and roundabouts. Finally, the Interchange Ramp Terminals chapter is now included with the interrupted-flow chapters.

Volume 3 is provided in both three-ring-binder and electronic formats to facilitate interim HCM updates as new research is performed.

VOLUME 4: APPLICATIONS GUIDE
Methodological Details
24. Concepts: Supplemental
25. Freeway Facilities: Supplemental
26. Freeway and Highway Segments: Supplemental
27. Freeway Weaving: Supplemental
28. Freeway Merges and Diverges: Supplemental
29. Urban Street Facilities: Supplemental
30. Urban Street Segments: Supplemental
31. Signalized Intersections: Supplemental
32. STOP-Controlled Intersections: Supplemental
33. Roundabouts: Supplemental
34. Interchange Ramp Terminals: Supplemental
35. Active Traffic Management
Interpretations
Case Studies
Technical Reference Library

VOLUME 4: APPLICATIONS GUIDE

Volume 4 is an electronic-only volume (www.HCM2010.org) that includes four types of content: supplemental chapters, methodological interpretations, comprehensive case studies, and a technical reference library.

The supplemental chapters include the following:

- More detailed descriptions of certain computational methodologies, written for users who seek a greater depth of understanding or plan to develop HCM implementation software;
- Example applications of alternative tools to situations not addressed by the Volume 2 or 3 chapter's methodology;
- Additional example problems and calculation results; and
- A new chapter on the impact of active traffic management techniques on roadway operations.

The methodological interpretations section will develop over time, as users apply the HCM 2010 and pose questions about particular methodologies to the Transportation Research Board (TRB) Committee on Highway Capacity and Quality of Service (AHB40). Clarifications of, interpretations of, and corrections

to the HCM that have been officially approved by the committee will be posted in the interpretations section of Volume 4.

Access Volume 4 at www.HCM2010.org

The comprehensive case studies illustrate how to use the HCM to perform common types of analyses. The case studies are focused on the process of applying the HCM, rather than on the details of performing calculations (which are addressed by the example problems). Case Studies 1 through 5 are derived from the web-based *HCM Applications Guidebook* (*6*) that was developed after the HCM2000 was published, while Case Study 6 was developed by the NCHRP 3-85 project (*7*).

The Technical Reference Library contains a selection of papers, technical reports, and companion documents that provide background information about the development of HCM methodologies.

COMPUTATIONAL ENGINES

Historically, all HCM methodologies have been fully documented within the manual through text, figures, and worksheets (the Freeway Facilities chapter in the HCM2000 represents the first departure from this pattern). However, in response to practitioner needs and identified HCM limitations, methodologies have continued to grow in complexity, and some have reached the point where they can no longer be feasibly documented in such a manner (for example, methodologies that require multiple iterations to reach a solution). In these cases, computational engines become an important means by which details of some of the more complex calculations can be fully described. For the most complex methodologies, the Volume 2 or 3 chapter, the Volume 4 supplemental chapter, and the computational engine together provide the most efficient and effective way of fully documenting the methodology.

HCM chapters describe, at a minimum, the process used by a given methodology. For simpler methodologies, the chapters fully describe the computational steps involved.

Supplemental chapters in Volume 4 provide calculation details for the more computationally complex methods.

Computational engines document all the calculation steps for the most complex methods, such as those involving iterative calculations.

The TRB Committee on Highway Capacity and Quality of Service maintains computational engines for most HCM methodologies for the purposes of evaluating methodologies as they are developed, developing new example problems, identifying needed improvements, and judging the impact of proposed changes. These engines are tools for developing and documenting HCM methodologies and do not have or need the sophisticated interfaces and input data manipulation techniques that would make them suitable for use in an engineering or planning office. The engines are not generally publicly distributed but are made available on request to researchers, practitioners, software developers, students, and others who are interested in understanding the inner workings of a particular HCM methodology.

COMMERCIAL SOFTWARE

To assist users in implementing the methodologies in the manual, commercial software is available (and has been since the publication of the 1985 HCM) to perform the numerical calculations for the more computationally intensive methods. A variety of commercial software products are available that implement HCM techniques and provide sophisticated user interfaces and data manipulation tools. It is the policy of TRB not to review or endorse commercial products.

4. INTERNATIONAL USE

APPLICATIONS

Capacity and quality-of-service analyses have generated interest on an international scale. The HCM has been translated into several languages, and research conducted in numerous countries outside of North America has contributed to the development of HCM methodologies. HCM users are cautioned, however, that the majority of the research base, the default values, and the typical applications are from North America, particularly from the United States. Although there is considerable value in the general methods presented, their use outside of North America requires additional emphasis on calibrating the equations and the procedures to local conditions, as well as recognizing major differences in the composition of traffic; in driver, pedestrian, and bicycle characteristics; and in typical geometrics and control measures.

METRIC CONVERSION GUIDE

The HCM2000 (*4*) was produced as two editions, one using U.S. customary units and the other using metric units. Variables in the HCM2000 were subject to *hard conversion,* meaning that figures were rounded where this was reasonable. For example, a lane width of 12 ft was converted to a rounded value of 3.6 m. In comparison, a soft conversion would multiply 12 ft by a conversion factor of 0.305 m/ft, resulting in a value of 3.66 m.

As described in this chapter's Structure section, there is no metric edition of the HCM 2010. Therefore, a soft conversion is favored from U.S. customary units to metric units, so that computational engines produce the same result regardless of the measurement system used. However, in comparisons of the metric results of methodologies that have not changed from 2000 to 2010, such as multilane highways, small discrepancies may appear: the results produced by the HCM 2010 with soft conversion may be slightly different from those produced by the metric version of the HCM2000.

Exhibit 1-1 provides approximate conversion factors from U.S. customary to metric units.

Exhibit 1-1
Metric Conversion Table

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in.	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
$in.^2$	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or metric tons)	Mg (or t)
		TEMPERATURE (exact conversion)		
°F	Fahrenheit	(F – 32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/square meter	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	pound force	4.45	newtons	N
$lbf/in.^2$	pound force per square inch	6.89	kilopascals	kPa

Source: Federal Highway Administration (*8*).

5. WHAT'S NEW IN THE HCM 2010

OVERVIEW

Although the page layout of the HCM 2010 is similar to that of the HCM2000, a number of changes have been made to the manual—a result of both the extensive research that has been conducted since the HCM2000 and the feedback received from HCM2000 users.

Research Basis for the HCM 2010

Exhibit 1-2 lists the major research projects that have contributed to the HCM 2010. The impacts of these and other projects on individual HCM chapters are described later in this section.

Exhibit 1-2
Major Research Projects Contributing to the HCM 2010

Project	Project Title	Project Objective(s)
NCHRP 3-60	Capacity and Quality of Service of Interchange Ramp Terminals	Develop improved methods for capacity and quality-of-service analysis of interchange ramp terminals, for a full range of interchange types.
NCHRP 3-64	*Highway Capacity Manual* Applications Guide	Develop an HCM Applications Guide that shows how to apply HCM methodologies to real-world problems and indicate when other methods may be more appropriate.
NCHRP 3-65	Applying Roundabouts in the United States	Develop methods of estimating the safety and operational impacts of U.S. roundabouts and refine the design criteria used for them.
NCHRP 3-70	Multimodal Level of Service Analysis for Urban Streets	Develop a framework and enhanced methods for determining levels of service for automobile, transit, bicycle, and pedestrian modes on urban streets, in particular with respect to the interaction among the modes.
NCHRP 3-75	Analysis of Freeway Weaving Sections	Develop improved methods for capacity and LOS analysis of freeway weaving sections.
NCHRP 3-79	Measuring and Predicting the Performance of Automobile Traffic on Urban Streets	Develop techniques to measure the performance of automobile traffic on urban streets for real-time applications; develop procedures to predict the performance of automobile traffic on urban streets.
NCHRP 3-82	Default Values for Capacity and Quality of Service Analyses	Determine appropriate default values for inputs to HCM analyses; develop a guide to select default values for various applications.
NCHRP 3-85	Guidance for the Use of Alternative Traffic Analysis Tools in Highway Capacity Analyses	Enhance the guidance in the HCM for the selection and use of alternative traffic analysis tools.
NCHRP 3-92	Production of the Year 2010 *Highway Capacity Manual*	Develop the 2010 edition of the HCM.
Federal Highway Administration	Evaluation of Safety, Design, and Operation of Shared-Use Paths (DTFH61-00-R-00070)	Develop an LOS estimation method for shared-use paths to assist path designers and operators in determining how wide to make new or rebuilt paths and whether to separate the different types of users.
Federal Highway Administration	Active Traffic Management Measures for Increasing Capacity and Improving Performance (DTFH61-06-D-00004)	Describe active traffic management techniques and available information and analysis methods for evaluating their effectiveness in increasing highway facility capacity and improving operational performance.

Focus Groups

After the publication of the HCM2000, the TRB Committee on Highway Capacity and Quality of Service sponsored a series of focus groups at various locations around the United States to obtain feedback and to identify desired improvements for the next edition. Committee and subcommittee members also prepared an audit of the HCM in the areas of planning, design and operations, and educational needs (*9*). After the HCM 2010 was funded, the Institute of Transportation Engineers sponsored a web-based survey on HCM usage and desired improvements, and the NCHRP 3-92 project organized several focus groups on those topics. The feedback from these and other sources was considered when decisions were made on the format, content, and organization of the HCM 2010.

Reorganization from the HCM2000

As described in detail in this chapter's Structure section, the HCM 2010 consists of four volumes: (*a*) Volume 1: Concepts, (*b*) Volume 2: Uninterrupted Flow, (*c*) Volume 3: Interrupted Flow, and (*d*) Volume 4: Applications Guide. Material from Parts I to V of the HCM2000 has been distributed to Volumes 1 to 4 of the HCM 2010 as follows:

- *Part I: Overview* material appears in Volume 1.
- *Part II: Concepts* material appears in Volumes 2 and 3 if used directly in an analysis (e.g., default values and LOS tables) and in Volume 1 otherwise.
- *Part III: Methodologies* material appears in Volume 2 for uninterrupted-flow chapters and Volume 3 for interrupted-flow chapters. Worksheets and highly detailed descriptions of methodological steps appear in the Volume 4 chapters.
- *Part IV: Corridor and Areawide* material that is conceptual in nature appears in Volume 1. More detailed analytical material has been removed in favor of guidance in the use of alternative tools for corridor and areawide analyses.
- *Part V: Simulation and Other Models* material has been distributed throughout the HCM 2010. Volume 1 contains an overview of alternative tools (Chapter 6) and general guidance on comparing HCM and alternative results (Chapter 7). Specific guidance on when to consider alternative tools is presented in each chapter in Volumes 2 and 3. Selected Volume 4 chapters provide examples of applying alternative tools to situations that cannot be addressed by HCM methodologies.

Multimodal Approach

To encourage HCM users to consider all travelers on a facility when they perform analyses and make decisions, the HCM 2010 integrates material on nonautomobile and automobile modes. Thus, there are no stand-alone Pedestrian, Bicycle, and Transit chapters in this edition. Instead, users should refer to the Urban Streets chapter for analysis procedures for pedestrians, bicyclists, and transit users on urban streets, to the Signalized Intersections chapter for procedures relating to signalized intersections, and so on.

In recognition of the companion TCQSM (*5*) and of the difficulty in keeping the two manuals in synch, users are referred to the TCQSM for transit-specific capacity and quality-of-service procedures. However, transit quality of service in a multimodal context continues to be addressed in the HCM.

Traveler-Perception Models

Since the 1985 HCM, LOS has been defined in terms of measures of operational conditions within a traffic stream (*3*, *4*). HCM methodologies have generally presented a single LOS measure per system element that can be (*a*) directly measured in the field, (*b*) perceived by travelers, and (*c*) affected by facility owners. However, since the publication of the HCM2000, a number of research projects have studied whether a single operational factor is sufficient to describe LOS, as well as whether nonoperational factors should also be used (*10*). These projects have proposed models that (*a*) incorporate multiple factors of traveler satisfaction and (*b*) set LOS thresholds based on traveler perceptions of service quality. Traveler-perception models from two of these studies (*11*, *12*) have been incorporated into the Multilane Highways, Two-Lane Highways, Urban Street Facilities, Urban Street Segments, and Off-Street Pedestrian and Bicycle Facilities chapters.

Generalized Service Volume Tables

The HCM2000 provided “example service volume tables” for 10 system elements. The service volume tables were developed by using a single set of default values and were accompanied by cautionary notes that they were illustrative only. The HCM 2010 provides “generalized service volume tables” for facilities that incorporate a range of national default values. These tables can be considered for such applications as statewide performance reporting, areawide (i.e., regional) modeling, and future-year analyses as part of a long-range transportation planning process.

METHODOLOGICAL CHANGES BY SYSTEM ELEMENT

Freeway Facilities

The basic methodology is similar to the one given in the HCM2000 but incorporates the new weaving-segment analysis procedure. A significant change is the addition of LOS thresholds for freeway facilities based on density. Other changes include updates to the material on the impact of weather and work zones on freeway facility capacity, along with new information on the impact of active traffic management measures on freeway operations.

Basic Freeway Segments

The basic methodology is similar to the one given in the HCM2000. The free-flow speed prediction model has been improved, and a speed–flow curve for segments with 75-mi/h free-flow speeds has been added.

Freeway Weaving Segments

This chapter has been completely updated and incorporates the methodology developed by the NCHRP 3-75 project. Although the general process for analyzing weaving segments is similar to that given in the HCM2000, the HCM 2010 models are based on an up-to-date set of weaving data. The following are the two major differences in how the methodology is applied: (*a*) there is now a single algorithm for predicting weaving speeds and a single algorithm for predicting nonweaving speeds, regardless of the weaving configuration, and (*b*) the LOS F threshold has changed.

Ramps and Ramp Junctions

The following revisions have been made to the HCM2000 methodology:

- Procedures have been added to check for unreasonable lane distributions that overload the left or right lane(s) (or both) of the freeway.
- A revision has been made to correct an illogical trend involving on-ramps on eight-lane freeways in which density increases as the length of the acceleration lane increases.

Multilane Highways

The multilane highways automobile methodology is essentially the same as that given in the HCM2000. A methodology for calculating bicycle LOS for multilane highways has been added.

Two-Lane Highways

The following revisions have been made to the HCM2000 automobile methodology:

- The two-direction analysis has been dropped: the one-direction methodology is the only one used, with two-direction results obtained by appropriate weighted averaging of the one-direction results.
- Several key curves and tables used in one-direction analyses have been adjusted and incorporated into the chapter.

A bicycle LOS methodology for two-lane highways has been added.

Urban Street Facilities

This is a new chapter that contains guidance to help analysts determine the scope of their analysis (i.e., isolated intersection versus coordinated signal system) and the relevant travel modes (i.e., automobile, pedestrian, bicycle, transit, or a combination). The methodology section describes how to aggregate results from the segment and point levels of analysis into an overall facility assessment. Information on the impact of active traffic management measures on urban street performance has been added.

Urban Street Segments

This chapter has been completely rewritten. The work of the NCHRP 3-79 project has been incorporated into the chapter, providing improved methods for estimating urban street free-flow speeds and running times, along with a new method for estimating the stop rate along an urban street. In addition, the work of the NCHRP 3-70 project has been incorporated, providing a multimodal LOS methodology that can be used to evaluate trade-offs in how urban street right-of-way is allocated among the modes using the street.

Signalized Intersections

The following revisions have been made to the HCM2000 methodology:

- A new incremental queue accumulation method has been added to calculate the d_1 delay term and the Q_1 length term. It is equivalent to the HCM2000 method for the idealized case but is more flexible to accommodate nonideal cases, including coordinated arrivals and multiple green periods with differing saturation flow rates (i.e., protected-plus-permitted left turns and sneakers).
- An actuated controller operation modeling procedure has been added.
- A left-turn lane overflow check procedure has been added.
- Pedestrian and bicycle LOS methodologies relating to signalized intersections have been moved into this chapter.

Unsignalized Intersections

The HCM2000's Unsignalized Intersections chapter has been split into three chapters: two-way STOP-controlled intersections, all-way STOP-controlled intersections, and roundabouts.

Two-Way STOP-Controlled Intersections

The two-way STOP-controlled intersection methodology for the automobile mode is essentially the same as the one given in the HCM2000, except gap-acceptance parameters for six-lane streets have been added. In addition, pedestrian and bicycle LOS methodologies relating to two-way STOP-controlled intersections have been moved into this chapter.

All-Way STOP-Controlled Intersections

The all-way STOP-controlled intersection methodology is essentially the same as the one given in the HCM2000. A queue-estimation model has been added.

Roundabouts

This chapter replaces the HCM2000 roundabout content. It is based on the work of the NCHRP 3-65 project, which developed a comprehensive database of U.S. roundabout operations and new methodologies for evaluating roundabout performance. A LOS table for roundabouts has been added.

Interchange Ramp Terminals

This chapter is completely updated on the basis of the NCHRP 3-60 project.

Off-Street Pedestrian and Bicycle Facilities

The pedestrian path procedures are essentially the same as those of the HCM2000, but guidance is provided on how to apply the procedures to a wider variety of facility types. The bicycle path procedures, which were based on Dutch research in the HCM2000, have been updated on the basis of results of a Federal Highway Administration (FHWA) study to calibrate the Dutch model for U.S. conditions and increase the number of path user groups (e.g., inline skaters and runners) addressed by the procedures.

6. COMPANION DOCUMENTS

Throughout its 60-year history, the HCM has been one of the fundamental reference works used by transportation engineers and planners. However, it is but one of a number of documents that play a role in the planning, design, and operation of transportation facilities and services. The HCM's scope is to provide tools to evaluate the performance of highway and street facilities in terms of operational and quality-of-service measures. This section describes companion documents to the HCM that cover important topics outside the HCM's scope.

HIGHWAY SAFETY MANUAL

The *Highway Safety Manual* (HSM) (*13*) provides analytical tools and techniques for quantifying the safety effects of decisions related to planning, design, operations, and maintenance. The information in the HSM is provided to assist agencies as they integrate safety into their decision-making processes. It is a nationally used resource document intended to help transportation professionals conduct safety analyses in a technically sound and consistent manner, thereby improving decisions made on the basis of safety performance.

A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS

The American Association of State Highway and Transportation Officials' *A Policy on Geometric Design of Highways and Streets* ("Green Book") (*14*) provides design guidelines for roadways ranging from local streets to freeways, in both urban and rural locations. The guidelines "are intended to provide operational efficiency, comfort, safety, and convenience for the motorist" and to emphasize the need to consider other modal users of roadway facilities.

MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES

FHWA's *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (*15*) is the national standard for traffic control devices for any street, highway, or bicycle trail open to public travel. Of particular interest to HCM users are the sections of the MUTCD pertaining to warrants for all-way STOP control and traffic signal control, signing and markings to designate lanes at intersections, and associated considerations of adequate roadway capacity and less restrictive intersection treatments.

TRANSIT CAPACITY AND QUALITY OF SERVICE MANUAL

The TCQSM (*5*) is the transit counterpart to the HCM. The manual contains background, statistics, and graphics on the various types of public transportation, and it provides a framework for measuring transit availability, comfort, and convenience from the passenger point of view. The manual contains quantitative techniques for calculating the capacity of bus, rail, and ferry transit services and transit stops, stations, and terminals.

7. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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CHAPTER 2
APPLICATIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

The *Highway Capacity Manual* (HCM) can be applied to transportation applications that range from the highly detailed to the highly generalized, to roadway system elements that range from individual points to an entire transportation system, to four travel modes that can be considered separately or in combination, and to several types of roadway and facility operating conditions. **Chapter 2, Applications,** introduces the wide range of potential HCM applications, all of which can be applied as stand-alone analyses or in support of a broader process.

Types of HCM analysis levels.

The HCM can be applied at the *operational, design, preliminary engineering,* and *planning* analysis levels. The required input data typically remain the same at each analysis level, but the degree to which analysis inputs use default values instead of actual measured or forecast values differs. In addition, operational analyses and planning and preliminary engineering analyses frequently evaluate the level of service (LOS) that will result from a given set of inputs, whereas design analyses typically determine which facility characteristics will be needed to achieve a desired LOS.

Travel modes and roadway system elements addressed by the HCM.

The travel modes covered by the HCM are the *automobile* [including other motorized vehicles such as trucks, recreational vehicles (RVs), intercity buses, and motorcycles], *pedestrian,* and *bicycle* modes, as well as *public transit* vehicles that operate on urban streets. All of these modes operate on a variety of roadway system elements, including *points* (e.g., intersections), *segments* (e.g., lengths of roadways between intersections), *facilities* (an aggregation of points and segments into longer lengths), *corridors* (parallel freeway and arterial facilities), and at the largest geographic scales, *areas* and *systems.*

HCM methodologies are provided both for *uninterrupted-flow* facilities, which have no fixed causes of delay or interruption external to the traffic stream, and for *interrupted-flow* facilities, on which traffic control devices such as traffic signals and STOP signs periodically interrupt the traffic stream. HCM analyses are applicable to *undersaturated* conditions (where demand is less than a roadway system element's capacity) and, in certain situations, to *oversaturated* conditions (where demand exceeds capacity).

Individual methodological chapters describe the extent to which the HCM can be used for oversaturated analyses. Chapter 6 describes alternative analysis tools that may be applied in situations in which the HCM cannot be used.

Finally, measures generated by HCM methodologies can be used for more than just stand-alone traffic analyses. This chapter describes potential applications of HCM methodologies to noise, air quality, economic, and multimodal planning analyses.

2. LEVELS OF ANALYSIS

OVERVIEW

Any given HCM application can be analyzed at different levels of detail, depending on the purpose of the analysis and the amount of information available. The HCM defines three primary levels of analysis:

- *Operational analysis,* typically focusing on current or near-term conditions, involving detailed inputs to HCM procedures, with no or minimal use of default values;
- *Design analysis,* typically using HCM procedures to identify the required characteristics of a transportation facility that will allow it to operate at a desired LOS, with some use of default values; and
- *Planning and preliminary engineering analysis,* typically focusing on future conditions, where it is desired to evaluate a series of alternatives quickly or when specific input values to procedures are not known, requiring the extensive use of default values.

The following sections describe these analysis levels further.

OPERATIONAL ANALYSIS

Operational analyses are applications of the HCM that are generally oriented toward current or near-term conditions. They aim at providing information for decisions on whether there is a need for improvements to an existing point, segment, or facility. Occasionally, an analysis is made to determine whether a more extensive planning study is needed. Sometimes the focus is on a network, or part of one, that is approaching oversaturation or an undesirable LOS: When, in the near term, is the facility likely to fail (or fail to meet a desired LOS threshold)? To answer this question, an estimate of the service flow rate allowable under a specified LOS is required.

The concept of LOS is described in Chapter 5, Quality and Level-of-Service Concepts.

HCM analyses also help practitioners make decisions about operating conditions. Typical alternatives often involve the analysis of appropriate lane configurations, alternative traffic control devices, signal timing and phasing, spacing and location of bus stops, frequency of bus service, and addition of a managed (e.g., high-occupancy vehicle) lane or a bicycle lane. The analysis produces operational measures for a comparison of the alternatives.

Because of the immediate, short-term focus of operational analyses, it is possible to provide detailed inputs to the models. Many of the inputs may be based on field measurements of traffic, physical features, and control parameters. Generally, it is inappropriate to use default values at this level of analysis.

DESIGN ANALYSIS

Design analyses primarily apply the HCM to establish the detailed physical features that will allow a new or modified facility to operate at a desired LOS. Design projects are usually targeted for mid- to long-term implementation. Not all the physical features that a designer must determine are reflected in the HCM models. Typically, analysts using the HCM seek to determine such elements as

the basic number of lanes required and the need for auxiliary or turning lanes. However, an analyst can also use the HCM to establish values for elements such as lane width, steepness of grade, length of added lanes, size of pedestrian queuing areas, widths of sidewalks and walkways, and presence of bus turnouts.

The data required for design analyses are fairly detailed and are based substantially on proposed design attributes. However, the intermediate to long-term focus of the work will require use of some default values. This simplification is justified in part by the limits on the accuracy and precision of the traffic predictions with which the analyst is working.

PLANNING AND PRELIMINARY ENGINEERING ANALYSIS

Planning analyses are applications of the HCM generally directed toward broad issues such as initial problem identification (e.g., screening a large number of locations for potential operations deficiencies), long-range analyses, and statewide performance monitoring. An analyst often must estimate the future times at which the operation of the current and committed systems will fall below a desired LOS. Preliminary engineering analyses are often conducted to support planning decisions related to roadway design concept and scope, and when alternatives analyses are performed. These studies can also assess proposed systemic policies, such as lane-use control for heavy vehicles, systemwide freeway ramp metering and other intelligent transportation system applications, and the use of demand-management techniques (e.g., congestion pricing) (*1*).

Planning and preliminary engineering analyses typically involve situations in which not all of the data needed for the analysis are available. Therefore, both types of analyses frequently rely on default values for many analysis inputs. Planning analyses may default nearly all inputs—for example, through the use of generalized service volume tables. Preliminary engineering analyses will typically fall between planning and design analyses in the use of default values.

RELATIONSHIP BETWEEN ANALYSIS LEVELS AND OBJECTIVES

Each methodological chapter in Volumes 2 and 3 has one basic method adapted to facilitate each of the levels of analysis. Analysis objectives include identifying a future problem, selecting an appropriate countermeasure to an identified problem, or evaluating the postimplementation success of an action. The HCM is particularly useful when a current situation is being studied in the context of future conditions or when an entirely new element of the system is being considered for implementation. Analysts studying current conditions should make direct field measurements of the performance attributes; these direct measurements can then be applied in the same manner as predicted values to determine performance measures of interest.

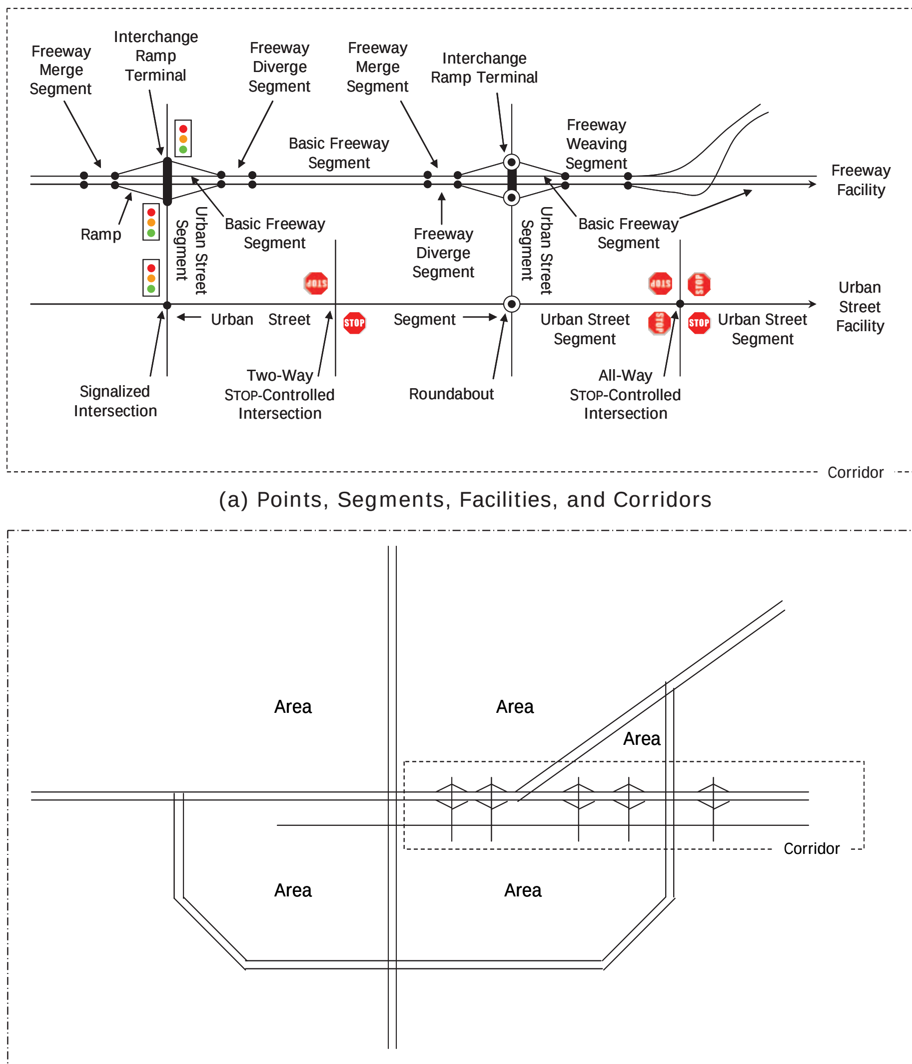
3. ROADWAY SYSTEM ELEMENTS

TYPES OF ROADWAY SYSTEM ELEMENTS

The HCM defines six main types of roadway system elements. From smallest to largest, these are points, segments, facilities, corridors, areas, and systems. The focus of the HCM is on the first three types of elements: points, segments, and facilities. Exhibit 2-1 illustrates the spatial relationships of these elements, and the following sections provide details about each system element type.

Exhibit 2-1
Illustrative Roadway System Elements

Note that a two-way STOP-controlled intersection does not normally divide the uncontrolled urban street into two segments.



(a) Points, Segments, Facilities, and Corridors

(b) Corridors, Areas, and Systems

Points

Points are places along a facility where (*a*) conflicting traffic streams cross, merge, or diverge; (*b*) a single traffic stream is regulated by a traffic control device; or (*c*) there is a significant change in the segment capacity (e.g., lane drop, lane addition, narrow bridge, significant upgrade, start or end of a ramp influence area).

Some points, such as interchange ramp terminals, may actually have a significant physical length associated with them, as suggested by Exhibit 2-1(a). For urban street facility analysis, points are treated as having zero length—all of the delay occurs at the point. For freeway facility analysis, points are used to define the endpoints of segments, but they have no associated performance measures or capacity, as these are calculated at the segment level.

Urban street points have a physical length but are treated as having zero length for facility analysis purposes.

Freeway points are used only to define the endpoints of segments—performance measures and capacity are not defined for them.

Segments

A segment is the length of roadway between two points. Traffic volumes and physical characteristics generally remain the same over the length of a segment, although small variations may occur (e.g., changes in traffic volumes on a segment resulting from a low-volume driveway). Segments may or may not be directional. The HCM defines basic freeway segments, freeway weaving segments, freeway merge and diverge segments, and urban street segments.

Facilities

Facilities are lengths of roadways, bicycle paths, and pedestrian walkways composed of a connected series of points and segments. Facilities may or may not be directional and are defined by two endpoints. The HCM defines freeway facilities, multilane highway facilities, two-lane highway facilities, urban street facilities, and pedestrian and bicycle facilities.

The types of facilities addressed by the HCM are described in Chapter 3, Modal Characteristics.

Corridors

Corridors are generally a set of parallel transportation facilities designed to move people between two locations. For example, a corridor may consist of a freeway facility and one or more parallel urban street facilities. There may also be rail or bus transit service on the freeway, the urban streets, or both, and transit service could be provided within a separate, parallel right-of-way. Pedestrian or bicycle facilities may also be present within the corridor, as designated portions of roadways and as exclusive, parallel facilities.

Areas

Areas consist of an interconnected set of transportation facilities serving movements within a specified geographic space, as well as movements to and from adjoining areas. The primary factor distinguishing areas from corridors is that the facilities within an area need not be parallel to each other. Area boundaries can be set by significant transportation facilities, political boundaries, or topographic features such as ridgelines or major bodies of water.

Systems

Systems are composed of all the transportation facilities and modes within a particular region. A large metropolitan area typically has multiple corridors passing through it, which divide the system into a number of smaller areas. Each area contains a number of facilities, which, in turn, are composed of a series of points and segments. Systems can also be divided into modal subsystems (e.g., the roadway subsystem, the transit subsystem) as well as subsystems composed of specific roadway elements (e.g., the freeway subsystem, the urban street subsystem).

ANALYSIS OF INDIVIDUAL SYSTEM ELEMENTS

The HCM provides tools to help analysts estimate performance measures for individual elements of a multimodal transportation system, as well as guidance on combining those elements to evaluate larger portions of the system. Exhibit 2-2 tabulates the various system elements for which the HCM provides analysis methodologies in Volumes 2 and 3, the service measure(s) used to determine LOS for each mode operating on each system element, and the HCM performance measure that can be used to aggregate results to a system level. Some combinations of system elements and travel modes combine several performance measures into a single traveler-perception model that is used to generate a LOS score; the components of each model are listed in Exhibit 2-3.

Exhibit 2-2
HCM Service Measures by System Element and Mode

System Element	HCM Chapter	Service Measure(s): Automobile	Pedestrian	Bicycle	Transit	Systems Analysis Measure
Freeway facility	10	Density	--	--	--	Speed
Basic freeway segment	11	Density	--	--	--	Speed
Freeway weaving segment	12	Density	--	--	--	Speed
Freeway merge and diverge segments	13	Density	--	--	--	Speed
Multilane highway	14	Density	--	LOS score[a]	--	Speed
Two-lane highway	15	Percent time-spent-following, speed	--	LOS score[a]	--	Speed
Urban street facility	16	Speed	LOS score[a]	LOS score[a]	LOS score[a]	Speed
Urban street segment	17	Speed	LOS score[a]	LOS score[a]	LOS score[a]	Speed
Signalized intersection	18	Delay	LOS score[a]	LOS score[a]	--	Delay
Two-way stop	19	Delay	Delay	--	--	Delay
All-way stop	20	Delay	--	--	--	Delay
Roundabout	21	Delay	--	--	--	Delay
Interchange ramp terminal	22	Delay	--	--	--	Delay
Off-street pedestrian–bicycle facility	23	--	Space, events[b]	LOS score[a]	--	Speed

Notes: [a] See Exhibit 2-3 for the LOS score components.
[b] Events are situations where pedestrians meet bicyclists.

Exhibit 2-3
Components of Traveler-Perception Models Used in the HCM

The automobile traveler-perception model for urban street segments and facilities is not used to determine LOS, but it is included to facilitate multimodal analyses.

System Element	HCM Chapter	Mode	Model Components
Multilane and two-lane highways	14, 15	Bicycle	Pavement quality, perceived separation from motor vehicles, motor vehicle volume and speed
Urban street facility	16	Automobile	Weighted average of segment automobile LOS scores
		Pedestrian	Urban street segment and signalized intersection pedestrian LOS scores, midblock crossing difficulty
		Bicycle	Urban street segment and signalized intersection bicycle LOS scores, driveway conflicts
		Transit	Weighted average of segment transit LOS scores
Urban street segment	17	Automobile	Stops per mile, left-turn lane presence
		Pedestrian	Pedestrian density, sidewalk width, perceived separation from motor vehicles, motor vehicle volume and speed
		Bicycle	Perceived separation from motor vehicles, pavement quality, motor vehicle volume and speed
		Transit	Service frequency, perceived speed, pedestrian LOS
Signalized intersection	18	Pedestrian	Street crossing delay, pedestrian exposure to turning vehicle conflicts, crossing distance
		Bicycle	Perceived separation from motor vehicles, crossing distance
Off-street pedestrian–bicycle facility	23	Bicycle	Average meetings/minute, active passings/minute, path width, centerline presence, delayed passings

ASSESSMENT OF MULTIPLE FACILITIES

The analysis of a transportation system starts with estimates of delay at the point and segment level. Point delays arise from the effects of traffic control devices such as traffic signals and STOP signs. Segment delays combine the point delay incurred at the end of the segment with other delays incurred within the segment. Examples of the latter include delays caused by midblock turning activity into driveways, parking activity, and midblock pedestrian crossings. The HCM estimates segment speed instead of segment delay; however, segment speed can be converted into segment delay by using Equation 2-1:

$$D_i = AVO_i \times d_i \left(\frac{L_i}{S_i} - \frac{L_i}{S_{0i}} \right)$$

Equation 2-1

where

D_i = person-hours of delay on segment *i*,

AVO_i = average vehicle occupancy on segment *i* (passengers/vehicle),

d_i = vehicle demand on segment *i* (vehicles),

L_i = length of segment *i* (mi),

S_i = average vehicle speed on segment *i* (mi/h), and

S_{0i} = free-flow speed of segment *i* (mi/h).

Typically, only the segments that constitute the collector and arterial system are used to estimate system delay.

Segment delays are added together to obtain facility estimates, and the sum of the facility estimates yields subsystem estimates. Mean delays for each subsystem are then computed by dividing the total person-hours of delay by the total number of trips on the subsystem. Subsystem estimates of delay can be combined into total system estimates, but typically the results for each subsystem are reported separately.

SYSTEM PERFORMANCE MEASUREMENT

An increase in congestion on one system element may result in a shift of demand to other system elements. Therefore, estimating system delay is an iterative process. HCM techniques can be used to estimate the delay resulting from a given demand, but not the demand resulting from a given delay.

System performance must be measured in more than one dimension. When a single intersection is analyzed, it may suffice to compute only the peak-period delay; however, when a system is analyzed, the geographic extent, the duration of delay, and any shifts in demand among facilities and modes must also be considered (2).

Dimensions of system performance.

System performance can be measured in the following dimensions:

- *Quantity of service*—the number of person-miles and person-hours provided by the system;
- *Intensity of congestion*—the amount of congestion experienced by users of the system;
- *Duration of congestion*—the number of hours that congestion persists;
- *Extent of congestion*—the physical length of the congested system;
- *Variability*—the day-to-day variation in congestion; and
- *Accessibility*—the percentage of the populace able to complete a selected trip within a specified time.

Quantity of Service

Quantity of service measures the utilization of the transportation system in terms of the number of people using the system, the distance they travel (person-miles of travel, PMT), and the time they require to travel (person-hours of travel, PHT). Dividing the PMT by the PHT gives the mean trip speed for the system.

Intensity of Congestion

The intensity of congestion can be measured by using total person-hours of delay and mean trip speed. Other metrics, such as mean delay per person-trip, can also be used. In planning and preliminary engineering applications, intensity of congestion is sometimes measured in terms of volume-to-capacity ratio or demand-to-capacity ratio.

Duration of Congestion

A segment is congested if the demand exceeds the segment's discharge capacity.

The duration of congestion is measured in terms of the maximum amount of time that congestion occurs anywhere in the system. A segment is congested if the demand exceeds the segment's discharge capacity. Transit subsystem congestion can occur either when the passenger demand exceeds the capacity of the transit vehicles or when the need to move transit vehicles exceeds the vehicular capacity of the transit facility.

Extent of Congestion

The extent of congestion may be expressed in terms of the directional miles of facilities congested or—more meaningfully for the public—in terms of the maximum percentage of system miles congested at any one time.

Variability

Ideally, variability should be measured in terms of either (*a*) the probability of occurrence of, or (*b*) a confidence interval for, other aspects of congestion (intensity, duration, and extent). However, the state of the practice does not yet facilitate such a calculation. Instead, a measure of the sensitivity of the results to changes in the demand can be substituted until better methods for estimating variability become available. Various levels of demand are tested (such as a 5% increase or a 5% decrease), and the resulting effects on the intensity, duration, and extent of congestion are noted in terms of a percentage increase or decrease in their values. The sensitivity can be expressed in terms of elasticity by dividing the percentage change in output by the percentage change in demand. An elasticity greater than 1.0 means that the estimated congestion measure is highly sensitive to changes in demand.

Accessibility

Accessibility examines the effectiveness of the system from a perspective other than intensity. Accessibility can be expressed in terms of the percentage of trips (or persons) able to accomplish a certain goal—such as going from home to work—within a targeted travel time. Accessibility can also be defined in terms of a traveler's ability to get to and use a particular modal subsystem, such as transit. This definition is closer to the Americans with Disabilities Act's use of the term.

4. TRAVEL MODES

This section introduces the four major travel modes addressed by the HCM: automobile, pedestrian, bicycle, and transit. Chapter 3 provides details about the characteristics of each mode that are important for HCM analyses.

AUTOMOBILE

The automobile mode includes all motorized vehicles except transit vehicles.

The automobile mode includes all motor vehicle traffic using a roadway except transit vehicles. Thus, trucks, RVs, motorcycles, and tour buses are all considered members of the automobile mode for HCM analysis purposes. Certain vehicle types (e.g., trucks and RVs) have operating characteristics different from those of private automobiles; these characteristics are taken into account by HCM methodologies where needed. The HCM's LOS thresholds for the automobile mode are based on the perspective of automobile drivers and passengers. Therefore, the automobile LOS measures may not necessarily reflect the perspective of drivers of other types of motorized vehicles, especially trucks.

PEDESTRIAN

The pedestrian mode consists of travelers along a roadway or pedestrian facility making a journey (or at least part of their journey) on foot. Pedestrians walk at different speeds, depending on their age, their ability, and environmental characteristics (e.g., grades and climate); HCM procedures generally account for this variability. Sidewalks and pathways may be used by more than just foot-based traffic—for example, inline skaters and persons in wheelchairs—but the HCM's LOS thresholds reflect the perspective of persons making a walking journey.

BICYCLE

The bicycle mode consists of travelers on a roadway or pathway who are using a nonmotorized bicycle for their trip; bicycle LOS thresholds reflect their perspective. Mopeds and motorized scooters are not considered bicycles for HCM analysis purposes.

TRANSIT

The companion TCQSM provides capacity- and speed-estimation procedures for transit vehicles and additional LOS measures for transit passengers.

Urban roadways are often shared with public transit buses and, occasionally, with rail transit vehicles such as streetcars and light rail vehicles. Previous editions of the HCM have provided relatively extensive coverage of the transit mode. However, now that two editions of a companion *Transit Capacity and Quality of Service Manual* (TCQSM) (*3*) have been published, the HCM now addresses the transit mode only with respect to multimodal analyses of urban streets. The HCM's LOS measure for transit on urban streets reflects the perspective of transit users—both those already on transit vehicles operating on the street and those waiting for transit vehicles at stops or stations along the street.

5. OPERATING CONDITIONS

UNINTERRUPTED FLOW

Uninterrupted-flow facilities have no fixed causes of delay or interruption external to the traffic stream. Volume 2 of the HCM provides analysis methodologies for uninterrupted-flow facilities.

Freeways and their components operate under the purest form of uninterrupted flow. Not only are there no fixed interruptions to traffic flow, but access is controlled and limited to ramp locations. Multilane highways and two-lane highways can also operate under uninterrupted flow in long segments between points of fixed interruption. On multilane and two-lane highways, it is often necessary to examine points of fixed interruption (e.g., traffic signals) as well as uninterrupted-flow segments.

The traffic stream on uninterrupted-flow facilities is the result of individual vehicles interacting with each other and the facility's geometric characteristics. The pattern of flow is generally controlled only by the characteristics of the land uses that generate traffic that use the facility, although freeway management and operations strategies—such as ramp metering, freeway auxiliary lanes, truck lane restrictions, variable speed limits, and incident detection and clearance—can also influence traffic flow. Operations can also be affected by environmental conditions, such as weather or lighting, by pavement conditions, and by the occurrence of traffic incidents (*4*, *5*).

"Uninterrupted flow" describes the type of facility, not the quality of the traffic flow at any given time. A freeway experiencing extreme congestion, for example, is still an uninterrupted-flow facility because the causes of congestion are internal.

INTERRUPTED FLOW

Interrupted-flow facilities have fixed causes of periodic delay or interruption to the traffic stream, such as traffic signals and STOP signs. Urban streets are the most common form of this kind of facility. Exclusive pedestrian and bicycle facilities are also treated as interrupted flow, since they may occasionally intersect other streets at locations where pedestrians and bicyclists do not automatically receive the right-of-way. Volume 3 of the HCM provides analysis methodologies for interrupted-flow facilities.

The traffic flow patterns on an interrupted-flow facility are the result not only of vehicle interactions and the facility's geometric characteristics but also of the traffic control used at intersections and the frequency of access points to the facility. Traffic signals, for example, allow designated movements to occur only during certain portions of the signal cycle (and, therefore, only during certain portions of an hour). This control creates two significant outcomes. First, time becomes a factor affecting flow and capacity because the facility is not available for continuous use. Second, the traffic flow pattern is dictated by the type of control used. For instance, traffic signals create platoons of vehicles that travel along the facility as a group, with significant gaps between one platoon and the

next. In contrast, all-way STOP-controlled intersections and roundabouts discharge vehicles more randomly, creating small (but not necessarily usable) gaps in traffic at downstream locations (*4*, *6*).

Platoons created by a traffic signal tend to disperse as they become more distant from the intersection. Many factors influence how quickly a platoon disperses, including the running speed and the amount of traffic that enters and leaves the facility between signalized intersections. In general, traffic signal spacing greater than 2 mi is thought to be sufficient for allowing uninterrupted flow to exist at some point between the signals. In contrast, on two-lane roadways downstream of a STOP-controlled intersection or roundabout, platoons may redevelop as faster vehicles catch up to slower-moving vehicles, although this effect is again dependent on the amount of driveway traffic entering and leaving the roadway (*4*, *6*).

UNDERSATURATED FLOW

Traffic flow during the analysis period is specified as “undersaturated” when the following conditions are satisfied: (*a*) the arrival flow rate is lower than the capacity of a point or segment, (*b*) no residual queue remains from a prior breakdown of the facility, and (*c*) traffic flow is unaffected by downstream conditions.

Uninterrupted-flow facilities operating in a state of undersaturated flow will typically have travel speeds within 10% to 20% of the facility’s free-flow speed, even at high flow rates, assuming base conditions (e.g., level grades, standard lane widths). Furthermore, no queues would be expected to develop on the facility.

Free-flow speed is the average speed of traffic on a segment as volume and density approach zero.

On interrupted-flow facilities, queues form as a natural consequence of the interruptions to traffic flow created by traffic signals and STOP and YIELD signs. Therefore, travel speeds are typically 30% to 65% below the facility’s free-flow speed in undersaturated conditions. Individual cycle failures—where a vehicle has to wait through more than one green phase to be served—may occur at traffic signals under moderate- to high-volume conditions as a result of natural variations in the cycle-to-cycle arrival and service rate. Similarly, STOP- and YIELD-controlled approaches may experience short periods of significant queue buildup. However, as long as all of the demand on an intersection approach is served within a 15-min analysis period, including any residual demand from the prior period, the approach is considered to be undersaturated.

OVERSATURATED FLOW

Traffic flow during an analysis period is characterized as “oversaturated” when any of the following conditions is satisfied: (*a*) the arrival flow rate exceeds the capacity of a point or segment, (*b*) a queue created from a prior breakdown of a facility has not yet dissipated, or (*c*) traffic flow is affected by downstream conditions.

On uninterrupted-flow facilities, oversaturated conditions result from a bottleneck on the facility. During periods of oversaturation, queues form and extend backward from the bottleneck point. Traffic speeds and flows drop

significantly as a result of turbulence, and they can vary considerably, depending on the severity of the bottleneck. Freeway queues differ from queues at undersaturated signalized intersections in that they are not static or "standing." On freeways, vehicles move slowly through a queue, with periods of stopping and movement. Even after the demand at the back of the queue drops, it takes some time for the queue to dissipate because vehicles discharge from the queue at a slower rate than they do under free-flow conditions. Oversaturated conditions persist within the queue until the queue dissipates after a period of time during which demand flows are less than the capacity of the bottleneck, allowing the queue to discharge completely.

On interrupted-flow facilities, oversaturated conditions generate a queue that grows backward from the intersection at a rate faster than can be processed by the intersection over the analysis period. Oversaturated conditions persist after demand drops below capacity until the residual queue (i.e., the queue over and above what would be created by the intersection's traffic control) has dissipated. A queue generated by an oversaturated unsignalized intersection dissipates more gradually than is typically possible at a signalized intersection.

If an intersection approach or ramp meter cannot accommodate all of its demand, queues may back into upstream intersections, adversely affecting their performance. Similarly, if an interchange ramp terminal cannot accommodate all of its demand, queues may back onto the freeway, adversely affecting its performance.

QUEUE DISCHARGE FLOW

A third type of flow, queue discharge flow, is particularly relevant for uninterrupted-flow facilities. Queue discharge flow represents traffic flow that has just passed through a bottleneck and, in the absence of another bottleneck downstream, is accelerating back to the facility's free-flow speed. Queue discharge flow is characterized by relatively stable flow as long as the effects of another bottleneck downstream are not present.

On freeways, this flow type is generally defined within a narrow range of 2,000 to 2,300 passenger cars per hour per lane, with speeds typically ranging from 35 mi/h up to the free-flow speed of the freeway segment. Lower speeds are typically observed just downstream of the bottleneck. Depending on horizontal and vertical alignments, queue discharge flow usually accelerates back to the facility's free-flow speed within 0.5 to 1 mi downstream of the bottleneck. Studies suggest that the queue discharge flow rate from the bottleneck is lower than the maximum flows observed before breakdown.

Because interrupted-flow facilities naturally operate from queue-discharge conditions, the queue discharge flow on these facilities is equal to the saturation flow rate.

Chapter 4, Traffic Flow and Capacity Concepts, provides details about the characteristics of traffic flow during undersaturated, oversaturated, and queue discharge conditions.

6. HCM ANALYSIS AS PART OF A BROADER PROCESS

Since its first edition in 1950, the HCM has provided transportation analysts with the tools to estimate traffic operational measures such as speed, density, and delay. It also has provided insights and specific tools for estimating the effects of various traffic, roadway, and other conditions on the capacity of facilities. Over time, the calculated values from the HCM have increasingly been used in other transportation work. The practice of using estimated or calculated values from HCM work as the foundation for estimating user costs and benefits in terms of economic value and environmental changes (especially air and noise) is particularly pronounced in transportation priority programs and in the justification of projects. This section provides examples of how HCM outputs can be used as inputs to other types of analyses.

NOISE ANALYSIS

At the time this chapter was written, federal regulations specifying noise abatement criteria stated that "in predicting noise levels and assessing noise impacts, traffic characteristics which will yield the worst hourly traffic noise impact on a regular basis for the design year shall be used" [23 CFR 772.17(b)]. The "worst hour" is usually taken to mean the loudest hour, which does not necessarily coincide with the busiest hour, since vehicular noise levels are directly related to speed. Traffic conditions in which large trucks are at their daily peak and in which LOS E conditions exist typically represent the loudest hour (*7*).

AIR QUALITY ANALYSIS

The 1990 Clean Air Act Amendments required state and local agencies to develop accurate emission inventories as an integral part of their air quality management and transportation planning responsibilities. Vehicular emissions are a significant contributor to poor air quality; therefore, the U.S. Environmental Protection Agency (EPA) has developed analysis procedures and tools for estimating emissions from mobile sources such as motorized vehicles. One input into the emissions model is average vehicle speed, which can be entered at the link (i.e., length between successive ramps) level, if desired. EPA's model has been found to be sensitive to average vehicle speed (i.e., a 20% change in average vehicle speed resulted in a greater than 20% change in the emissions estimate), implying that accurate speed inputs are a requirement for accurate emissions estimates. The HCM is a tool recommended by EPA for generating speed estimates on freeways and arterials and collectors (*8–10*).

ECONOMIC ANALYSIS

The economic analysis of transportation improvements also depends to a large extent on information generated from the HCM. Road user benefits are directly related to reductions in travel time and delay, while costs are determined from construction of roadway improvements (e.g., addition of lanes, installation

of traffic signals) and increases in travel time and delay. The following excerpt (*11,* p. 3-2) indicates the degree to which such analyses depend on the HCM:

> *The [HCM] provides many tools and procedures to assist in the calculation of segment speeds. These procedures permit detailed consideration of segment features, including the effects of road geometry and weaving on the capacity and speed of a highway segment. Speed can be calculated for local streets and roads, highways and freeways using the [HCM]. The most accurate rendering of the effects of additional lanes on speed, therefore, is through the use of the [HCM] calculation procedures.*

MULTIMODAL PLANNING ANALYSIS

An increasing number of jurisdictions are taking an integrated approach to multimodal transportation planning. That is, rather than simply developing plans for the automobile, transit, and pedestrian and bicycle modes in isolation, these jurisdictions are evaluating trade-offs among the modes as part of their transportation planning and decision making. This edition of the HCM is designed to support those efforts. For example, Chapter 16, Urban Street Facilities, presents an integrated, multimodal set of LOS measures for urban streets. The other interrupted-flow chapters in Volume 3 also integrate pedestrian and bicycle measures, to the extent that research is available to support those measures.

SYSTEM PERFORMANCE MEASUREMENT

State and federal governments use HCM procedures in reporting transportation system performance. For example, the Federal Highway Administration's Highway Performance Monitoring System uses HCM procedures to estimate the capacity of highway sections and to determine volume-to-service flow ratios (*12*). Florida uses HCM procedures to estimate speeds on the state highway system as part of its mobility performance measures reporting.

SUMMARY

In summary, almost all economic analyses and all air and noise environmental analyses rely directly on one or more measures estimated or produced with HCM calculations. Exhibit 2-4 lists the automobile-based performance measures from this manual that are applicable to environmental or economic analyses.

Exhibit 2-4
HCM Automobile Performance Measures for Environmental and Economic Analyses

Chapter	Automobile Performance Measure	Analysis Types Appropriate for Use: Air	Noise	Economic
10. Freeway Facilities	Density*			
	Vehicle hours of delay			√
	Speed	√	√	√
	Travel time			√
11. Basic Freeway Segments	Density*			
	Speed	√	√	√
	v/c ratio	√		√
12. Freeway Weaving Segments	Density*			
	Weaving speed	√	√	√
	Nonweaving speed	√	√	√
13. Freeway Merge and Diverge Segments	Density*			
	Speed	√	√	√
14. Multilane Highways	Density*			
	Speed	√	√	√
	v/c ratio	√		√
15. Two-Lane Highways	Percent time-spent-following*			
	Speed*	√	√	√
16. Urban Street Facilities 17. Urban Street Segments	Speed*			
	Stop rate	√	√	√
	Running time	√		√
	Intersection control delay	√		√
18. Signalized Intersections 19. TWSC Intersections 20. AWSC Intersections 21. Roundabouts 22. Interchange Ramp Terminals	Control delay*	√		√
	v/c ratio	√		√

Notes: * Chapter service measure.
TWSC: two-way STOP-controlled, AWSC: all-way STOP-controlled, *v/c*: volume to capacity.

7. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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CHAPTER 3
MODAL CHARACTERISTICS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Roadways serve users of many different modes: motorists, pedestrians, bicyclists, and transit passengers. The roadway right-of-way is allocated among the modes through the provision of facilities that ideally serve each mode's needs. However, in many urban situations, the right-of-way is constrained by adjacent land development, causing transportation engineers and planners to consider trade-offs in how to allocate the right-of-way. Interactions among the modes that result from different right-of-way allocations are important to consider in analyzing a roadway, and the *Highway Capacity Manual* (HCM) provides tools for assessing these interactions. Local policies and design standards relating to roadway functional classifications are other sources of guidance on the allocation of right-of-way; safety and operational concerns should also be addressed.

Chapter 3, Modal Characteristics, introduces some basic characteristics of the four major modes addressed by the HCM. The following characteristics are considered in this chapter for each mode:

- Factors that contribute to a traveler's experience during a trip,
- Observed seasonal and daily variations in travel demand,
- Types of transportation facilities employed by a given mode,
- Notable capacity and volume observations, and
- Descriptions of the interactions that occur between modes.

Chapters 4 and 5 continue the discussion of multimodal performance. Chapter 4 discusses flow and capacity concepts and provides operational performance measures for each mode. Chapter 5 discusses quality and level-of-service (LOS) concepts and introduces the measures of effectiveness for each mode that the HCM uses to assess transportation facilities from a traveler point of view.

2. AUTOMOBILE MODE

VEHICLE AND HUMAN FACTORS

Three major elements affect driving: the vehicle, the roadway environment, and the driver. This section identifies vehicle and driver characteristics and how they are affected by the roadway's environment and physical properties.

Motor Vehicle Characteristics

Characteristics of various motorized roadway users.

This section provides a summary of the operating characteristics of motor vehicles that should be considered when a facility is analyzed. The major considerations are vehicle types and dimensions, turning radii and off-tracking, resistance to motion, power requirements, acceleration performance, and deceleration performance. Motor vehicles include passenger cars, trucks, vans, buses, recreational vehicles, and motorcycles. All of these vehicles have unique weight, length, size, and operational characteristics.

Vehicle acceleration and deceleration rates are factors that must be considered in designing traffic signal timing, computing fuel economy and travel time, and estimating how normal traffic flow resumes after a breakdown. Passenger cars accelerate after a stop at a rate ranging between 5.5 and 9 ft/s^2. Trucks, however, accelerate from a stop at rates ranging from 0.7 to 3 ft/s^2. Maximum passenger car deceleration rates range between 10 and 25 ft/s^2, depending on road surface and tire conditions, with deceleration rates of 10 ft/s^2 or less considered reasonably comfortable for passenger car occupants (*1*). Typical truck deceleration rates are 6.5 ft/s^2 or lower (*2*).

Driver Characteristics

Driving is a complex task involving a variety of skills. The most important skills are taking in and processing information and making quick decisions on the basis of this information. Driver tasks are grouped into three main categories: control, guidance, and navigation. Control involves the driver's interaction with the vehicle in terms of speed and direction (accelerating, braking, and steering). Guidance refers to maintaining a safe path and keeping the vehicle in the proper lane. Navigation means planning and executing a trip.

The way in which drivers perceive and process information is important. About 90% of information is presented to drivers visually. The speed at which drivers process information is a significant component affecting their successful use of the information. One parameter used to quantify the speed at which drivers process information is perception–reaction time, which represents how quickly drivers can respond to an emergency situation. Another parameter—sight distance—is directly associated with reaction time. There are three types of sight distance: stopping, passing, and decision. Sight distance is a parameter that helps determine appropriate geometric features of transportation facilities. Acceptance of gaps in traffic streams is associated with driver perception and influences the capacity and delay of movements at unsignalized intersections.

Factors such as nighttime driving, fatigue, driving under the influence of alcohol and drugs, the age and health of drivers, and police enforcement also contribute to driver behavior on a transportation facility. All these factors can affect the operational parameters of speed, delay, and density. However, unless otherwise specified, the HCM assumes base conditions of daylight, dry pavement, typical drivers, and so forth.

Base conditions are discussed generally in Chapter 4 and specifically in chapters in Volumes 2 and 3.

VARIATIONS IN DEMAND

The traffic volume counted at a given location on a given day is not necessarily reflective of the amount of traffic (*a*) that would be counted on another day or (*b*) that would be counted if an upstream bottleneck was removed. Traffic demand varies seasonally, by day of the week (e.g., weekdays versus weekends), and by hour of the day, as trip purposes and the number of persons desiring to travel varies. Bottlenecks—locations where the capacity provided is insufficient to meet the demand during a 15-min or hourly period—constrain the observed volume to the portion of the demand that can be served by the bottleneck. Because traffic counts only provide the portion of the demand that was served, the actual demand can be difficult to identify.

Demand relates to the number of vehicles that would like to be served by a roadway element, while volume relates to the number that can actually be served.

The following sections discuss monthly, daily, and hourly variations in traffic demand. Analysts need to account for these types of variations to ensure that the peak-hour demand volumes used in an HCM analysis are reflective of conditions on peak days of the year. Failure to account for these variations can result in an analysis that reflects peak conditions on the days counts were made, but not peak conditions over the course of the year. For example, a highway serving a beach resort area may be virtually unused during much of the year but become oversaturated during the peak summer periods.

Seasonal peaks in traffic demand must also be considered, particularly on recreational facilities.

Chapter 4, Traffic Flow and Capacity Concepts, discusses subhourly variations in demand. It is possible for a roadway's capacity to be greater than its hourly demand, yet traffic flow may still break down if the flow rate within the hour exceeds the roadway's capacity. The effects of a breakdown can extend far beyond the time during which demand exceeded capacity and may take several hours to dissipate.

A highway that is barely able to handle a peak-hour demand may be subject to breakdown if flow rates within the peak hour exceed capacity—a topic of Chapter 4.

The data shown in the exhibits in this section represent typical observations that can be made. The patterns illustrated, however, vary in response to local travel habits and environments, and these examples should not be used as a substitute for locally obtained data.

Data shown in these graphs represent typical observations but should not be used as a substitute for local data.

Seasonal and Monthly Variations

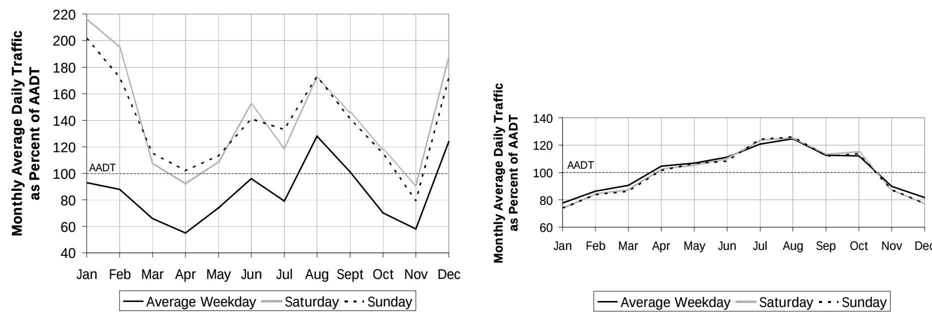
Seasonal fluctuations in traffic demand reflect the social and economic activity of the area served by the highway. Exhibit 3-1 shows monthly patterns observed in Oregon and Washington. The highway depicted in Exhibit 3-1(a) serves national forestland with both winter and summer recreational activity. The highway depicted in Exhibit 3-1(b) is a rural route serving intercity traffic. Two significant characteristics are apparent from this data set:

- The range of variation in traffic demand over the course of a year is more severe on rural routes primarily serving recreational traffic than on rural routes primarily serving intercity traffic; and
- Traffic patterns vary more severely by month on recreational routes.

Exhibit 3-1
Examples of Monthly Traffic Volume Variations for a Highway

Monthly volume variations for routes with recreational traffic show much higher seasonal peaking than for routes with predominantly intercity traffic.

The average daily traffic averaged over a full year is referred to as the annual average daily traffic, or AADT, and is often used in forecasting and planning.



(a) Routes with Significant Recreational Traffic (b) Routes with Significant Intercity Traffic

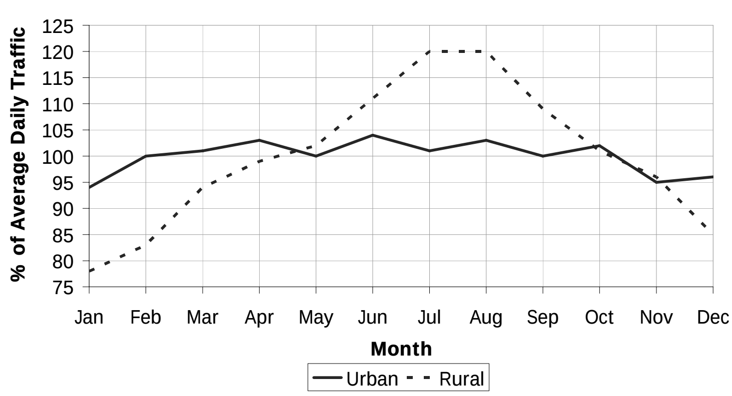
Note: (a) Highway 35 south of Parkdale, Oregon; (b) US-97 north of Wenatchee, Washington.
Source: (a) Oregon DOT, 2007; (b) Washington State DOT, 2007.

These and other similar observations lead to the conclusion that commuter- and business-oriented travel occurs in fairly uniform patterns, while recreational traffic creates the greatest variation in demand patterns.

The data for Exhibit 3-2 were collected on the same Interstate route. One segment is within 1 mi of the central business district of a large metropolitan area. The other segment is within 75 mi of the first but serves a combination of recreational and intercity travel. This exhibit illustrates that monthly variations in volume are more severe on rural routes than on urban routes. The wide variation in seasonal patterns for the two segments underscores the effect of trip purpose and may also reflect capacity restrictions on the urban section.

Exhibit 3-2
Examples of Monthly Traffic Volume Variations for the Same Interstate Highway (Rural and Urban Segments)

Monthly volume variations for rural segments of Interstate highways show much higher seasonal peaking than for urban segments of the same highway. This may reflect both recreational and agricultural traffic impacts.

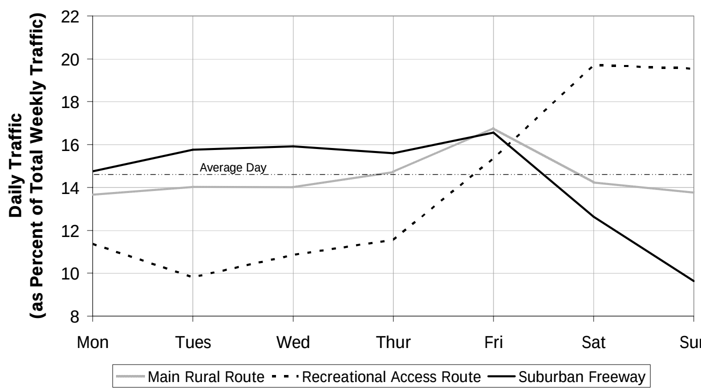


Note: Urban, I-84 east of I-5 in Portland; rural, I-84 at Rowena.
Source: Oregon DOT, 2006.

Daily Variations

Demand variations by day of the week are also related to the type of highway. Exhibit 3-3 shows that weekend volumes are lower than weekday volumes for highways serving predominantly business travel, such as urban freeways. In comparison, peak traffic occurs on weekends on main rural and recreational highways. Furthermore, the magnitude of daily variation is highest for recreational access routes and lowest for urban commuter routes.

Time of peak demand will vary according to highway type.

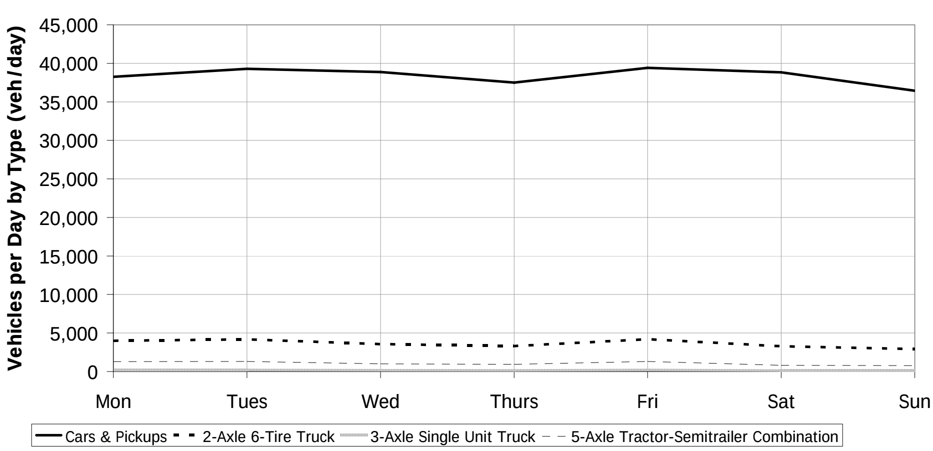


Notes: Suburban freeway, I-182 in Richland, Washington; main rural route, US-12 southeast of Pasco, Washington; recreational access route, Highway 35 south of Parkdale, Oregon.
Source: Washington State DOT, 2007, and Oregon DOT, 2007.

Exhibit 3-3
Examples of Daily Traffic Variation by Type of Route

Daily volume variations through the week show higher weekday volumes and lower weekend volumes for routes primarily serving commuter and intercity traffic, but the opposite for segments serving recreational traffic. Fridays are typically the peak weekday.

Exhibit 3-4 shows the variation in traffic by vehicle type for the shoulder lane of an urban freeway. Although the values shown in Exhibit 3-3 and Exhibit 3-4 are typical of patterns that may be observed, they should not be used as a substitute for local studies and analyses.



Note: Northbound Highway 16 north of I-5, Tacoma, Washington.
Source: Washington State DOT, 2007.

Exhibit 3-4
Daily Variation in Traffic by Vehicle Type for the Right Lane of an Urban Freeway

Daily volume variations by vehicle type through the week show higher weekday volumes and lower weekend volumes for truck traffic, with much sharper drops on the weekend for heavy truck traffic then for single-unit trucks. Car and pickup traffic peaks on Fridays and declines much more mildly on weekends on this urban freeway.

Hourly Variations

Typical hourly variation patterns for rural routes are shown in Exhibit 3-5, where the patterns are related to highway type and day of the week. Unlike urban routes, rural routes tend to have a single peak that occurs in the afternoon. A small morning peak is visible on weekdays that is much lower than the afternoon peak. The proportion of daily traffic occurring in the peak hour is much higher for recreational access routes than for intercity or local rural routes.

The weekend pattern for recreational routes is similar to the weekday pattern, as travelers tend to go to their recreation destination in the morning and return in the later afternoon. Weekend morning travel is considerably lower than weekday morning travel for the other types of rural routes.

Exhibit 3-5
Examples of Hourly Traffic Variations for Rural Routes

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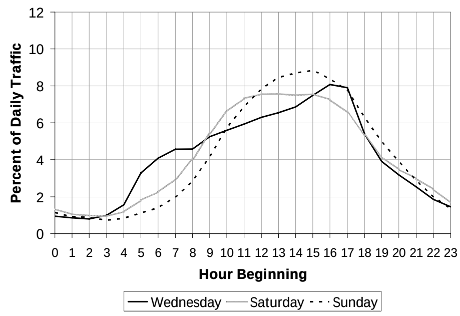
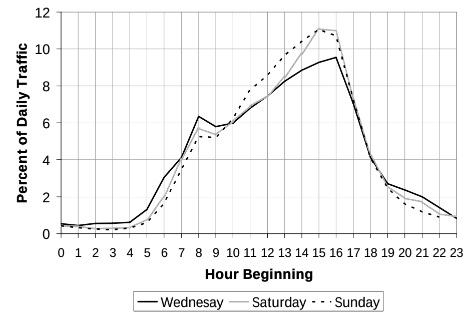
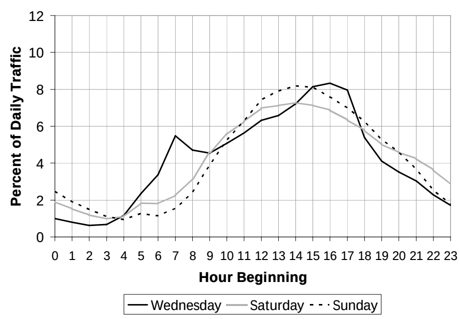
Traffic variation during the day by day of week for rural routes.



(a) Intercity Route



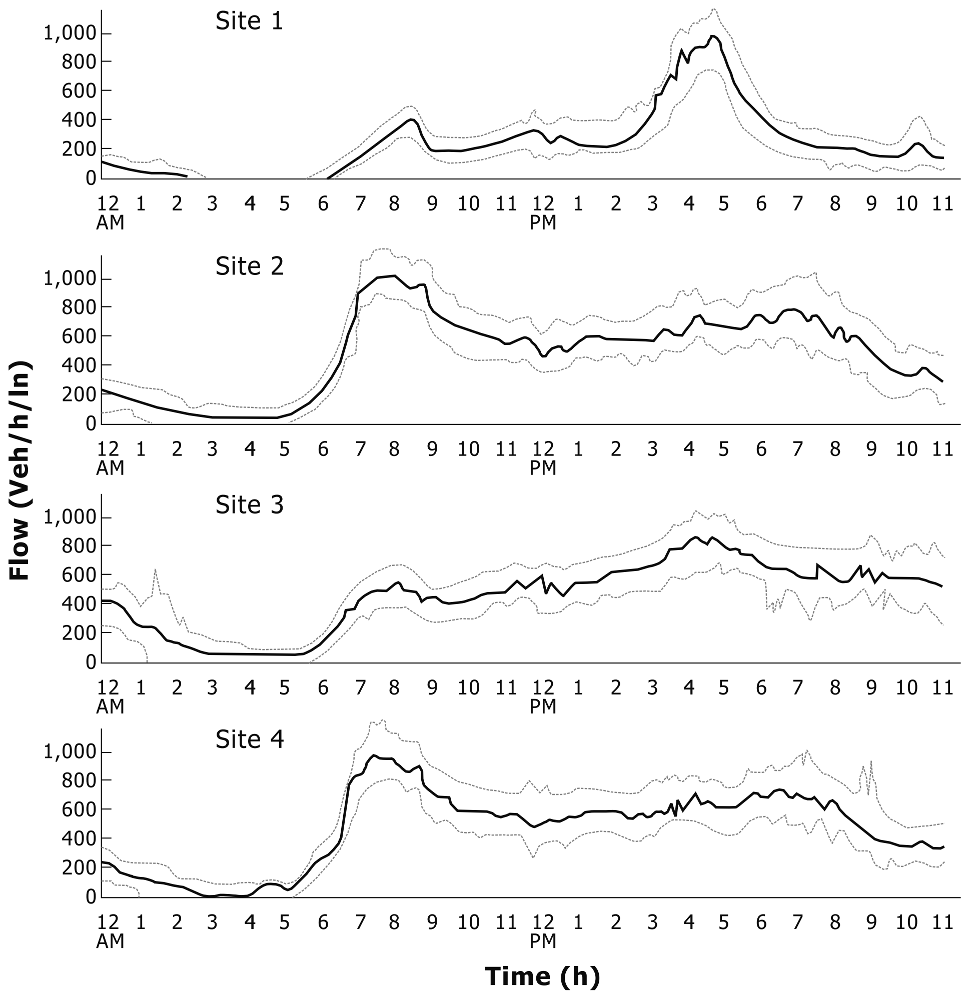
(b) Recreation Access Route



(c) Local Route

Notes: (a) US-395 south of Kennewick, Washington; (b) Highway 35 south of Parkdale, Oregon; (c) US-97 near Wapato, Washington.
Source: Washington State DOT, 2007, and Oregon DOT, 2007.

The repeatability of hourly variations is of great importance. The stability of peak-hour demand affects the feasibility of using such values in design and operational analyses of highways and other transportation facilities. Exhibit 3-6 shows data obtained for single directions of urban streets in metropolitan Toronto. The data were obtained from detectors measuring traffic in one direction only, as evidenced by the single peak period shown for either morning or afternoon. The area between the dotted lines indicates the range within which one can expect 95% of the observations to fall. Whereas the variations by hour of the day are typical for urban areas, the relatively narrow and parallel fluctuations among the days of the study indicate the repeatability of the basic pattern.



Notes: Sites 2 and 4 are one block apart on the same street, in the same direction. All sites are two moving lanes in one direction.
Source: McShane and Crowley (*3*).

Exhibit 3-6
Repeatability of Hourly Traffic Variations for Urban Streets

Repeatability of hourly patterns.

Peak Hour and Analysis Hour

Capacity and other traffic analyses typically focus on the peak-hour traffic volume because it represents the most critical period for operations and has the highest capacity requirements. However, as shown in the previous sections, the peak-hour volume is not a constant value from day to day or from season to season.

If the highest hourly volumes for a given location were listed in descending order, the data would vary greatly, depending on the type of facility. Rural and recreational routes often show a wide variation in peak-hour volumes. Several extremely high volumes occur on a few select weekends or in other peak periods, and traffic during the rest of the year flows at much lower volumes, even during the peak hour. Urban streets, on the other hand, show less variation in peak-hour traffic. Most users are daily commuters or frequent users, and occasional and special event traffic is minimal. Furthermore, many urban routes are filled to capacity during each peak hour, and variation is therefore severely constrained—an issue that will be revisited later in this section.

Exhibit 3-7 shows hourly volume relationships measured on four highway types in Washington. The recreational highway shows the widest variation in peak-hour traffic. Its values range from 25% of AADT in the highest hour of the year to about 16.3% of AADT in the 200th-highest hour of the year. The main rural freeway also varies widely, with 17.3% of the AADT in the highest hour, decreasing to 10.8% in the 200th-highest hour. The urban freeways show far less variation. The range in percent of AADT covers a narrow band, from approximately 9.7% (radial freeway) and 7.3% (circumferential freeway) for the highest hour to 8.9% and 6.9%, respectively, for the 200th-highest hour. Exhibit 3-7 is based on all hours of the year, not just peak hours of each day, and shows only the highest 200 hours of the year.

Exhibit 3-7
Ranked Hourly Volumes

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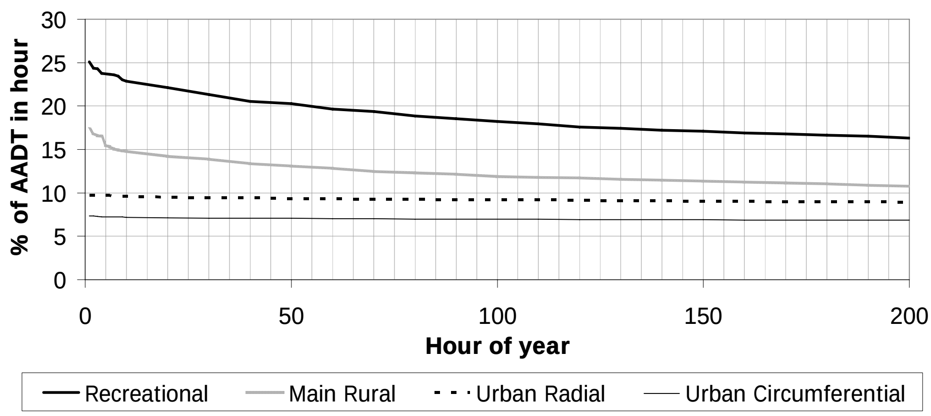


Notes: Recreational, US-2 near Stevens Pass (AADT = 3,862); main rural, I-90 near Moses Lake (AADT = 10,533); urban radial, I-90 in Seattle (AADT = 120,173); urban circumferential, I-405 in Bellevue (AADT = 141,550).
Source: Washington State DOT, 2006.

Selection of an analysis hour usually implies that a small portion of the demand during a year will not be adequately served.

The selection of an appropriate hour for planning, design, and operational purposes is a compromise between providing an adequate LOS for every (or almost every) hour of the year and providing economic efficiency. Customary practice in the United States is to base rural highway design on the 30th-highest hour of the year. There are few hours with higher volumes than this hour, while there are many hours with volumes not much lower. In urban areas, there is usually little difference between the 30th- and 200th-highest hours of the year, because of the recurring morning and afternoon commute patterns (*4*).

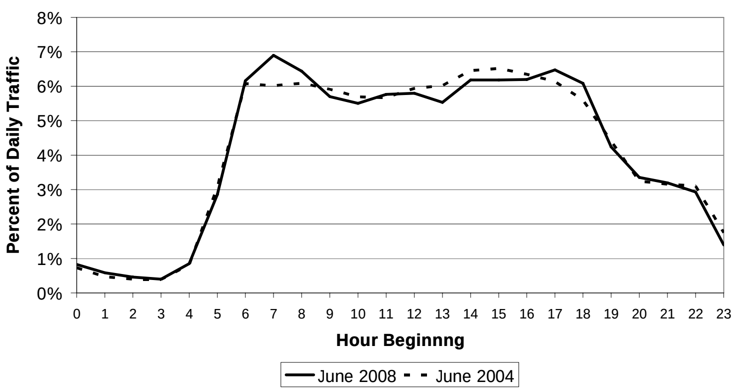
The selection of the analysis hour should consider the impact on the design and operations of higher-volume hours that are not accommodated. The recreational access route curve of Exhibit 3-7 shows that the highest hours of the year have one-third more volume than the 100th-highest hour, whereas the highest hours of an urban radial route were only about 6% higher than the volume in the 100th-highest hour. Use of a design criterion set at the 100th-highest hour would create substantial congestion on a recreational access route during the highest-volume hours but would have less effect on an urban facility. Another consideration is the LOS objective. A route designed to operate at LOS C can absorb larger amounts of additional traffic than a route designed to operate at LOS D or E during the hours of the year operating with higher volumes than the design hour. As a general guide, the most frequently occurring peak volumes

may be considered in the design of new or upgraded facilities. The LOS during higher-volume periods should be tested to determine the acceptability of the resulting traffic conditions.

Measured traffic volume patterns may not reflect actual demand patterns.

On roadways where oversaturation occurs during peak periods, analysts should be particularly careful in selecting a design hour, since measured traffic volumes may not reflect the changes in demand that occur once a bottleneck is removed. Exhibit 3-8 shows hourly variations in traffic on an urban freeway before and after the freeway was widened. In the before condition, the freeway's observed volumes were constrained by a bottleneck between 6 and 10 a.m., as indicated by the flat volume line. After the freeway widening, a more typical a.m. peak occurred, since travel patterns more closely reflected when travelers desired to travel rather than when the freeway could accommodate their travel.

Exhibit 3-8
Example of a Change in Travel Patterns Following Removal of a Capacity Constraint



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Note: I-25 south of US-6, Denver.
Source: Colorado DOT.

As used in the HCM, the *K*-factor is the proportion of AADT that occurs during the peak hour. For many rural and urban highways, this factor falls between 0.09 and 0.10. For highway sections with high peak periods and relatively low off-peak flows, the *K*-factor may exceed 0.10. Conversely, for highways that demonstrate consistent and heavy flows for many hours of the day, the *K*-factor is likely to be lower than 0.09. In general,

- The *K*-factor decreases as the AADT on a highway increases;
- The *K*-factor decreases as development density increases; and
- The highest *K*-factors occur on recreational facilities, followed by rural, suburban, and urban facilities, in descending order.

The *K*-factor should be determined, if possible, from local data for similar facilities with similar demand characteristics.

Exhibit 3-9 demonstrates how *K*-factors decrease as AADT increases, on the basis of average data from Washington State.

Exhibit 3-9
Example *K*-Factors by AADT

AADT	Average *K*-Factor	Number of Sites Included in Average *K*-Factor: Urban	Recreational	Other Rural
0–2,500	0.151	0	6	12
2,500–5,000	0.136	1	6	8
5,000–10,000	0.118	2	2	14
10,000–20,000	0.116	1	2	15
20,000–50,000	0.107	11	5	10
50,000–100,000	0.091	14	0	4
100,000–200,000	0.082	11	0	0
>200,000	0.067	2	0	0

Note: *K*-factors are for the 30th-highest traffic volume hour of the year.
Source: Washington State DOT (*5*).

Spatial Distributions

Traffic volume varies in space as well as time. The two critical spatial characteristics used to analyze capacity are directional distribution and volume distribution by lane. Volume may also vary longitudinally along various segments of a facility, but this does not explicitly affect HCM analyses because each facility segment that serves a different traffic demand is analyzed separately.

D-Factor

Concept of D-factor or directional distribution.

The *D*-factor is the proportion of traffic moving in the peak direction of travel on a given roadway during the peak hours. A radial route serving strong directional demands into a city in the morning and out at night may display a 2:1 imbalance in directional flows. Recreational and rural routes may also be subject to significant directional imbalances, which must be considered in analyses. Circumferential routes and routes connecting two major cities within a metropolitan area may have very balanced flows during peak hours. Exhibit 3-10 provides examples of directional distributions from selected California freeways.

Exhibit 3-10
Example Directional Distribution Characteristics

Freeway Type	*D*-Factor
Rural–intercity	0.59
Rural–recreational and intercity	0.64
Suburban circumferential	0.52
Suburban radial	0.60
Urban radial	0.70
Intraurban	0.51

Notes: Rural–intercity, I-5 at Willows; rural–recreational and intercity, I-80 west of Donner Summit; suburban circumferential, I-680 in Danville; suburban radial, I-80 in Pinole; urban radial, Highway 94 at I-5, San Diego; intraurban, I-880 in Hayward.
Source: Caltrans, 2007.

Directional distribution is an important factor in highway capacity analysis. This is particularly true for two-lane rural highways. Capacity and level of service vary substantially with directional distribution because of the interactive nature of directional flows on such facilities—the flow in one direction of travel influences flow in the other direction by affecting the number of passing opportunities. Procedures for two-lane highway analyses include explicit consideration of directional distribution.

While the consideration of directional distribution is not mandated in the analysis of multilane facilities, the distribution has a dramatic effect on both design and LOS. As indicated in Exhibit 3-10, up to two-thirds of the peak-hour traffic on urban radial routes has been observed as moving in one direction. Unfortunately, this peak occurs in one direction in the morning and in the opposite direction in the evening. Thus, both directions of the facility must have adequate capacity for the peak directional flow. This characteristic has led to the use of reversible lanes on some urban streets and highways.

Directional distribution is not a static characteristic. It changes annually, hourly, daily, and seasonally. Development in the vicinity of highway facilities often changes the directional distribution.

The *D*-factor is used with the *K*-factor to estimate the peak-hour traffic volume in the peak direction, as shown by Equation 3-1:

Equation 3-1

$$DDHV = AADT \times K \times D$$

where

$DDHV$ = directional design-hour volume (veh/h),

$AADT$ = annual average daily traffic (veh/day),

K = proportion of $AADT$ occurring in the peak hour (decimal), and

D = proportion of peak-hour traffic in the peak direction (decimal).

Lane Distribution

Concept of lane distribution.

When two or more lanes are available for traffic in a single direction, the lane-use distribution varies widely. The volume distribution by lane depends on traffic regulations, traffic composition, speed and volume, the number and location of access points, the origin–destination patterns of drivers, the development environment, and local driver habits.

Because of these factors, there are no typical lane distributions. Data indicate that the peak lane on a six-lane freeway, for example, may be the shoulder, middle, or median lane, depending on local conditions.

Exhibit 3-11 gives daily lane distribution data for various vehicle types on three selected freeways. These data are illustrative and are not intended to represent typical values.

Exhibit 3-11
Lane Distribution by Vehicle Type

Highway	Vehicle Type	Percent Distribution by Lane[a]		
		Lane 3	Lane 2	Lane 1
Lodge Freeway, Detroit	Light[b]	32.4	38.4	29.2
	Single-unit trucks	7.7	61.5	30.8
	Combinations	8.6	2.9	88.5
	All vehicles	31.3	37.8	30.9
I-95, Connecticut Turnpike	Light[b]	24.5	40.9	34.6
	All vehicles	22.5	40.4	37.1
I-4, Orlando, Florida	All vehicles	38.4	31.7	29.9

Notes: [a] Lane 1 = shoulder lane; lanes numbered from right to left.
[b] Passenger cars, panel trucks, and pickup trucks.
Sources: Huber and Tracy (*6*); Florida DOT, 1993.

The trend indicated in Exhibit 3-11 is reasonably consistent throughout North America. Heavier vehicles tend to use the right-hand lanes, partially because they operate at lower speeds than other vehicles and partially because regulations may prohibit them from using the leftmost lanes.

Lane distribution must also be considered at intersections, since it affects how efficiently the demand for a particular movement can be served, as well as lane-by-lane queue lengths. Uneven lane distributions can be a result of upstream or downstream changes in the number of lanes available and the pre-positioning of traffic for downstream turning movements.

TRAVEL TIME VARIABILITY

Travelers using the same set of roadways notice that the time required to make their trip may vary from one day to the next. Depending on the importance of reaching one's destination by a given time (e.g., to start work, to pick up a child at day care, to deliver a shipment), one may budget extra time for the trip to allow for the possibility of a longer-than-usual travel time. This variability in travel times has several sources (*7*):

- *Traffic incidents* such as crashes, stalled cars, and debris in the roadway block travel lanes, reducing the roadway's capacity. Even when lanes are not physically blocked, activity on the shoulder (e.g., police action) or in the opposite direction of travel (e.g., a crash scene being cleared) can lead to changes in driver behavior that result in congestion.
- *Work zones* may provide a reduced number of lanes or reduced clearances between vehicles (e.g., due to narrower lanes) and roadside objects (e.g., due to reduced or eliminated shoulders), which affect capacity. Speed limits may also be reduced in work zones. Temporary road closures may result in a diversion of traffic to other roadways, increasing traffic volumes on those roads above typical levels.
- *Environmental conditions* such as adverse weather, bright sunlight directly in drivers' eyes, and abrupt transitions from light to dark (such as at a tunnel entrance on a sunny day) may cause drivers to slow down and increase their spacing, resulting in a drop in a roadway's capacity.
- *Fluctuations in demand* occur both in longer-term patterns (by day of the week and by month of the year, as shown in Exhibit 3-1 through Exhibit 3-5) and more randomly from day to day, as shown in Exhibit 3-6. Variable traffic demand on a roadway with fixed capacity results in variable travel times.
- *Special events* are a special case of demand fluctuation occurring at known times relatively infrequently, resulting in traffic flow patterns that vary substantially from the typical situation.
- *Traffic control devices* that intermittently disrupt traffic flow (e.g., railroad crossings and drawbridges), as well as poorly timed traffic signals, contribute to travel time variability.
- *Inadequate base capacity*, interacting with the other six factors listed above, also influences travel time variability. Depending on how close a facility

is to operating at its capacity, relatively small changes in demand can result in disproportionately large changes in travel time. Facilities with a greater base capacity are less vulnerable to disruptions: a freeway with three lanes per direction that experiences an incident blocking one lane loses at least one-third of its directional capacity, while a freeway with two lanes per direction would lose at least one-half of its directional capacity.

AUTOMOBILE FACILITY TYPES

Exhibit 3-12 illustrates the kinds of automobile facilities addressed in the HCM. They are divided into two main categories: *uninterrupted-flow facilities,* where traffic has no fixed causes of delay or interruption beyond the traffic stream, and *interrupted-flow facilities,* where traffic control such as traffic signals and STOP signs introduce delay into the traffic stream.

Exhibit 3-12
Automobile Facility Types

(a) Freeway

(b) Multilane Highway

(c) Two-Lane Highway

(d) Urban Street

Uninterrupted Flow

Freeways are fully access-controlled, divided highways with a minimum of two lanes (and frequently more) in each direction. Certain lanes on freeways may be reserved for designated types of vehicles, such as high-occupancy vehicles or trucks. Some freeway facilities charge tolls, and their toll-collection facilities can create interrupted-flow conditions, such as on facilities where tolls are paid manually at toll plazas located on the freeway mainline. *Ramps* provide access to, from, and between freeways; some ramps have meters that control the flow of traffic onto a freeway segment.

Multilane highways are divided highways with a minimum of two lanes in each direction. They have zero or partial control of access. Traffic signals or roundabouts may create periodic interruptions to flow along an otherwise uninterrupted facility, but such interruptions are spaced at least 2 mi apart.

As the name implies, *two-lane highways* generally have a two-lane cross section, although passing and climbing lanes may be provided periodically. Within the two-lane sections, passing maneuvers must be made in the opposing lane. Traffic signals, STOP-controlled intersections, or roundabouts may occasionally interrupt flow, but at intervals longer than 2 mi.

Interrupted Flow

Urban streets are streets with relatively high densities of driveway and cross-street access, located within urban areas. The traffic flow of urban streets is interrupted (i.e., traffic signals, all-way stops, or roundabouts) at intervals of 2 mi or less. HCM procedures are applicable to arterial and collector urban streets, including those in downtown areas, but these procedures are not designed to address local streets.

MEASURED AND OBSERVED VOLUMES AND FLOW RATES

The direct observation of capacity is difficult to achieve for several reasons. The recording of a high, or even a maximum, volume or flow rate for a given facility does not ensure that a higher flow could not be accommodated at another time. Furthermore, capacity is sometimes an unstable operating condition. Depending on environmental factors, the mix of familiar and unfamiliar drivers in the traffic stream, and other considerations, the capacity achieved at a given location—or sets of otherwise similar locations—may vary from day to day.

HCM methodologies are based on calibrated national-average traffic characteristics observed over a range of facilities. Observations of these characteristics at specific locations will vary somewhat from national averages because of unique features of the local driving environment.

Freeways

HCM freeway capacity analysis procedures use a flow rate of 2,400 passenger cars per hour per lane (pc/h/ln) for basic freeway segments with free-flow speeds of 70 to 75 mi/h and 2,300 pc/h/ln for basic freeway segments with free-flow speeds of 65 mi/h as the capacity under base conditions.

Multilane Highways

The observation of multilane rural highways operating under capacity conditions is difficult, because such operations rarely occur. The HCM uses a flow rate of 2,200 pc/h/ln for multilane highways with free-flow speeds of 60 mi/h and 2,100 pc/h/ln for multilane highways with free-flow speeds of 55 mi/h as the capacity under base conditions.

Rural Two-Way, Two-Lane Highways

Two-way, two-lane rural highways in the United States and Canada rarely operate at volumes near capacity, and thus the observation of capacity

operations for such highways in the field is difficult. The HCM methodology reports *single-direction* capacities, with a flow rate of 1,700 pc/h/ln used as the capacity under base conditions.

Urban Streets

Since flow on urban streets is uninterrupted only on segments between intersections, the interpretation of high-volume observations on urban streets is not as straightforward as for uninterrupted-flow facilities. Signal timing significantly alters the capacity of such facilities by limiting the time that is available for movement along the urban street at critical intersections. The prevailing conditions on urban streets may vary greatly, and such factors as curb parking, transit buses, lane widths, and upstream intersections may substantially affect operations and observed volumes.

INTERACTIONS WITH OTHER MODES

Each mode that uses a roadway interacts with the other modal users of that roadway. This section examines the effects of other modes on the automobile mode; the effects of the automobile mode on other modes are discussed later in the portions of the chapter addressing those modes.

Pedestrians

Pedestrians interact with the automobile mode on interrupted-flow elements of the roadway system. At signalized intersections, the minimum green time provided for an intersection approach is influenced by the need to provide adequate time for pedestrians using the parallel crosswalk to cross the roadway safely. In turn, the green time allocated to a particular vehicular movement affects the capacity of and the delay experienced by that movement. At signalized and unsignalized intersections, turning vehicles must yield to pedestrians in crosswalks, which reduces the capacity of and increases the delay to those turning movements, compared with a situation in which pedestrians are not present. The increased delays at intersections and midblock pedestrian crossings along urban streets that result from higher pedestrian crossing volumes lower vehicular speeds along the urban street.

Bicycles

At intersections, automobile capacity and delay are affected by bicycle volumes, particularly where turning vehicles conflict with through bicycle movements. However, HCM methodologies only account for these effects at signalized intersections. Bicycles may also delay automobiles on two-lane roadways, in cases where bicycles use the travel lane, causing vehicles to wait for a safe opportunity to pass. This kind of delay is not currently accounted for in the HCM two-lane roadway methodology, which only addresses delays associated with waiting to pass other automobiles.

Transit

Transit vehicles are longer than automobiles and have different performance characteristics; thus, they are treated as heavy vehicles for all types of roadway

elements. At intersections, buses or streetcars that stop in the vehicular travel lane to serve passengers delay other vehicles in the lane and reduce the lane's capacity; however, this effect is only incorporated into the signalized intersection methodology. Special transit phases or bus signal priority measures at signalized intersections affect the allocation of green time to the various traffic movements, with accompanying effects on vehicular capacity and delay.

3. PEDESTRIAN MODE

OVERVIEW

Approximately 9% of all trips in the United States are accomplished by walking (2001 National Household Travel Survey). Moreover, many automobile trips and most transit trips include at least one section of the trip where the traveler is a pedestrian. Given the presence of a network of safe and convenient pedestrian facilities, as well as the availability of potential destinations within walking distance of one's trip origin, walking can be the mode of choice for a variety of shorter trips, including going to school, running errands, and recreational and exercise trips.

HUMAN FACTORS

A pedestrian is considerably more exposed than is a motorist, in both good and bad ways. A pedestrian travels much more slowly than other modal users and can therefore pay more attention to his or her surroundings. The ability to take in one's surroundings and get exercise while doing so can be part of the enjoyment of the trip. At the same time, a pedestrian interacts closely with other modal users, including other pedestrians, with potential safety, comfort, travel hindrance, and other implications. In addition, a pedestrian is exposed to the elements. As a result, a number of environmental and perceived safety factors significantly influence pedestrian quality of service. In locations with large numbers of pedestrians, pedestrian flow quality is also a consideration.

Some pedestrian flow measures are similar to those used for vehicular flow, such as the freedom to choose desired speeds and to bypass others. Others are related specifically to pedestrian flow, such as (*a*) the ability to cross a pedestrian traffic stream, to walk in the reverse direction of a major pedestrian flow, and to maneuver without conflicts or changes in walking speed and (*b*) the delay experienced by pedestrians at signalized and unsignalized intersections.

Environmental factors also contribute to the walking experience and, therefore, to the quality of service perceived by pedestrians. These factors include the comfort, convenience, safety, security, and economics of the walkway system. Comfort factors include weather protection; proximity, volume, and speed of motor vehicle traffic; pathway surface; and pedestrian amenities. Convenience factors include walking distances, intersection delays, pathway directness, grades, sidewalk ramps, wayfinding signage and maps, and other features making pedestrian travel easy and uncomplicated.

Safety is provided by separating pedestrians from vehicular traffic both horizontally, by using pedestrian zones and other vehicle-free areas, and vertically, by using overpasses and underpasses. Traffic control devices such as pedestrian signals can provide time separation of pedestrian and vehicular traffic, which improves pedestrian safety. Security features include lighting, open lines of sight, and the degree and type of street activity.

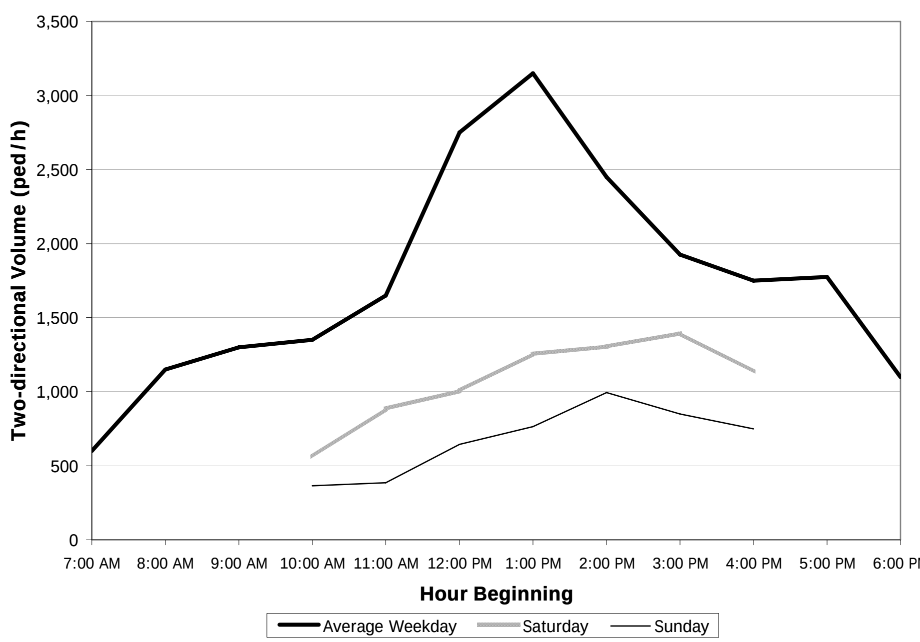
The economics of pedestrian facilities relate to user costs brought about by travel delays and inconvenience and to commercial values and retail development influenced by pedestrian accessibility.

Chapter 4, Traffic Flow and Capacity Concepts, discusses pedestrian flow measures, such as speed, space, and delay, while Chapter 5, Quality and Level-of-Service Concepts, covers the environmental factors that influence pedestrian quality of service.

VARIATIONS IN DEMAND

Pedestrian demand differs from the other modes addressed in the HCM in that the peak pedestrian demand often occurs at midday or during the early afternoon. Depending on the location, secondary peaks or plateaus in demand may also occur during the weekday a.m. and p.m. peak hours. Exhibit 3-13 shows two-directional pedestrian volume data collected in May 2004 on a sidewalk in Lower Manhattan, for an average of five weekdays in a week, Saturday, and Sunday. Although weekday demand was considerably higher than weekend demand, a single peak can be seen clearly in all three counts. Work-related trips made up the majority of a.m. peak-period pedestrian trips, while non-work-related and tourist trips made up the majority of the midday and early afternoon pedestrian trips (*8*).

Exhibit 3-13
Temporal Variations in Pedestrian Demand



Source: Adapted from New York City Department of City Planning (*8*).

PEDESTRIAN FACILITY TYPES

Exhibit 3-14 illustrates the types of pedestrian facilities addressed in the HCM. The following sections define each type of facility.

Exhibit 3-14
Pedestrian Facility Types


(a) Sidewalk


(b) Walkway


(c) Pedestrian Zone


(d) Queuing Area


(e) Crosswalk


(f) Underpass


(g) Overpass


(h) Stairway


(i) Shared Pedestrian–Bicycle Path

Sidewalks, Walkways, and Pedestrian Zones

These three facility types are separated from motor vehicle traffic and typically are not designed for bicycles or other nonpedestrian users, other than persons in wheelchairs. They accommodate the highest volumes of pedestrians and provide the best levels of service, because pedestrians do not share the facility with other modes traveling at higher speeds.

Sidewalks are located parallel and in proximity to roadways. Pedestrian walkways are similar to sidewalks in construction and may be used to connect sidewalks, but they are located well away from the influence of automobile traffic. Pedestrian zones are streets that are dedicated to pedestrian use on a full- or part-time basis.

Pedestrian walkways are also used to connect portions of transit stations and terminals. Pedestrian expectations about speed and density in a transit context are different from those in a sidewalk context; the *Transit Capacity and Quality of Service Manual* (TCQSM) (*9*) provides more information on this topic.

Queuing Areas

Queuing areas are places where pedestrians stand temporarily while waiting to be served, such as at the corner of a signalized intersection. In dense standing crowds, there is little room to move, and circulation opportunities are limited as the average space per pedestrian decreases.

Pedestrian Crosswalks

Pedestrian crosswalks, whether marked or unmarked, provide connections between pedestrian facilities across sections of roadway used by automobiles, bicycles, and transit vehicles. Depending on the type of control used for the crosswalk, local laws, and driver observance of those laws, pedestrians will experience varying levels of delay, safety, and comfort while waiting to use the crosswalk.

Stairways

Stairways are sometimes used to help provide pedestrian connectivity in areas with steep hills, employing the public right-of-way that would otherwise contain a roadway. Today they are often also used in conjunction with a ramp or elevator to provide shorter access routes to overpasses, underpasses, or walkways located at a different elevation. Even a small number of pedestrians moving in the opposite direction of the primary flow can significantly decrease a stairway's capacity to serve the primary flow.

Overpasses and Underpasses

Overpasses and underpasses provide a grade-separated route for pedestrians to cross wide or high-speed roadways, railroad tracks, busways, and topographic features. Access is typically provided by a ramp or, occasionally, an elevator, which is often supplemented with stairs. Procedures exist to assess the quality of pedestrian flow on these facilities, but not the quality of the pedestrian environment.

Shared Pedestrian–Bicycle Paths

Shared pedestrian paths typically are open to use by nonmotorized modes such as bicycles, skateboards, and inline skaters. Shared-use paths often are constructed to serve areas without city streets and to provide recreational opportunities for the public. These paths are common on university campuses, where motor vehicle traffic and parking are often restricted. In the United States, there are few paths exclusively for pedestrians; most off-street paths, therefore, are for shared use.

On shared facilities, bicycles—because of their markedly higher speeds—can have a negative effect on pedestrian capacity and quality of service. However, it is difficult to establish a bicycle–pedestrian equivalent because the relationship between the two depends on the characteristics of the cycling population and the modes' respective flows, directional splits, and other factors.

INTERACTIONS WITH OTHER MODES

Automobiles

At signalized intersections, the delay experienced by pedestrians is influenced in part by the amount of green time allocated to serve vehicular volumes on the street being crossed. The volume of cars making turns across a crosswalk at an intersection also affects a pedestrian's delay and perception of the intersection's quality of service.

At unsignalized intersections, increased major-street traffic volumes affect pedestrian crossing delay by reducing the number of opportunities for pedestrians to cross. The effect of motor vehicle volumes on pedestrian delay at unsignalized intersections also depends on local laws specifying yielding requirements to pedestrians in crosswalks and driver observation of those laws.

Automobile and heavy vehicle traffic volumes, and the extent to which pedestrians are separated from vehicular traffic, influence pedestrians' perceptions of quality of service while using a sidewalk.

Bicycles

Bicycle interaction with pedestrians is greatest on pathways shared by the two modes. Bicycles—because of their markedly higher speeds—can have a negative effect on pedestrian capacity and quality of service on such pathways.

Transit

The interaction of transit vehicles with pedestrians is similar to that of automobiles. However, because transit vehicles are larger than automobiles, the effect of a single transit vehicle is proportionately greater than that of a single automobile. The lack of pedestrian facilities in the vicinity of transit stops can be a barrier to transit access, and transit quality of service is influenced by the quality of the pedestrian environment along streets with transit service. Although it is not addressed by the HCM procedures, the pedestrian environment along the streets used to get to and from the streets with transit service also influences transit quality of service. Passengers waiting for buses at a bus stop can reduce the effective width of a sidewalk, while passengers getting off buses may create cross-flows that interact with the flow of pedestrians along a sidewalk.

4. BICYCLE MODE

OVERVIEW

Bicycles are used to make a variety of trips, including trips for recreation and exercise, commutes to work and school, and trips for errands and visiting friends. Bicycles help extend the market area of transit service, since bicyclists can travel about five times as far as an average person can walk in the same amount of time. Although bicycle trip-making in North America is lower than in the rest of the world, several large North American cities that have invested in bicycle infrastructure and programs (e.g., Portland, Oregon; Minneapolis, Minnesota; and Vancouver, Canada) have bicycle commute mode splits around 4% (2005–2007 census and local data). Some college towns like Eugene, Oregon, and Boulder, Colorado, have commute mode splits exceeding 8% (2007 census data), and Davis, California, achieved a 14% mode split (2005–2007 census data).

HUMAN FACTORS

Many of the measures of vehicular effectiveness can also describe bicycling conditions, whether on exclusive or shared facilities. As with motor vehicles, bicycle speeds remain relatively insensitive to flow rates over a wide range of flows. Delays due to traffic control affect bicycle speeds along a facility, and the additional effort required to accelerate from a stop is particularly noticeable to bicyclists. Grades, bicycle gearing, and the bicyclist's fitness level also affect bicycle speed and the level of effort required to maintain a particular speed.

Some vehicular measures are less applicable to the bicycle mode. For example, bicycle density is difficult to assess, particularly with regard to facilities shared with pedestrians and others. Because of the severe deterioration of service quality at flow levels well below capacity (e.g., freedom to maneuver around other bicyclists), the concept of capacity has little utility in the design and analysis of bicycle paths and other facilities. Capacity is rarely observed on bicycle facilities; rather, cyclists typically dismount and walk their bicycles before a facility reaches capacity. Values for capacity therefore reflect sparse data, generally from European studies or from simulation.

Hindrance as a performance measure.

Other measures of bicycle quality of service have no vehicular counterpart. For example, the concept of hindrance relates directly to bicyclists' comfort and convenience (*10*). During travel on a bicycle facility, two significant parameters can be easily observed and identified: (*a*) the number of users (other bicyclists, pedestrians, etc.) moving in the same direction and passed by the bicyclist and (*b*) the number of users moving in the opposing direction encountered by the bicyclist. Each event causes some discomfort and inconvenience to the bicyclist.

As is the case with pedestrians, environmental factors contribute significantly to the bicycling experience and, therefore, to quality of service. These factors include the volume and speed of adjacent vehicles, heavy vehicle presence, the presence of on-street parking, and the quality of the pavement. Chapter 5, Quality and Level-of-Service Concepts, discusses environmental and hindrance factors, while Chapter 4, Traffic Flow and Capacity Concepts, presents bicycle flow measures.

VARIATIONS IN DEMAND

Bicycle travel demand varies by time of day, day of the week, and month of the year. All of these variations in demand are related to trip-making demands in general (e.g., bicycle demand is highest during weekday a.m. and p.m. peak periods, just as with motor vehicles). Bicyclists are more exposed than motorists to the elements and other roadway users. Dutch research has shown that weather explains up to 80% of annual variation in bicycle travel, with higher rainfall and lower temperatures resulting in lower rates of bicycling (*11*).

Therefore, monthly variations in bicycle demands are also weather- and daylight-related. Exhibit 3-15 shows variations in bicycle demand hourly, daily, and monthly, on the basis of data from Copenhagen, Denmark, and Portland, Oregon. Exhibit 3-16 shows observations of bicycle demand compared with variations in daily high temperature along a bicycle path in Boulder, Colorado.

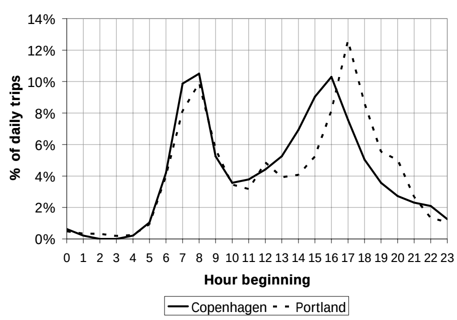
Exhibit 3-15
Temporal Variations in Bicycle Demand

Copenhagen and Portland hourly demand patterns are nearly identical through noon. In the afternoon, Copenhagen peaks an hour earlier (due to a shorter workday length). The Portland p.m. peak is noticeably higher than the a.m. peak, unlike Copenhagen.

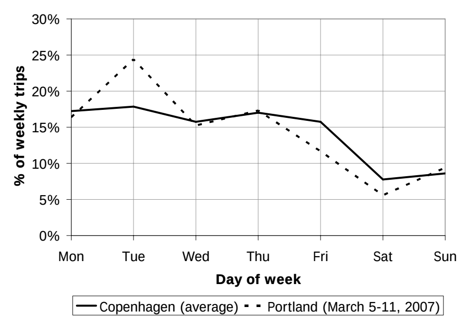
Bicycle volumes can fluctuate significantly from day to day, as suggested by the Portland line on the weekly variation chart. The Portland volumes may reflect the effects of rain on Wednesday, Friday, Saturday, and Sunday of that week.

Automobile volumes also drop in July in Copenhagen, when many people go on vacation. Except for July, months with higher temperatures and more daylight have higher bicycle volumes in Copenhagen.

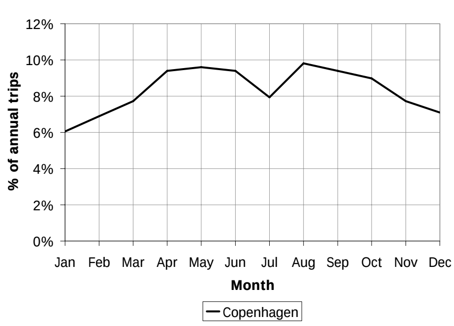
LIVE GRAPH
Click here to view



(a) Hourly Variations



(b) Weekly Variations

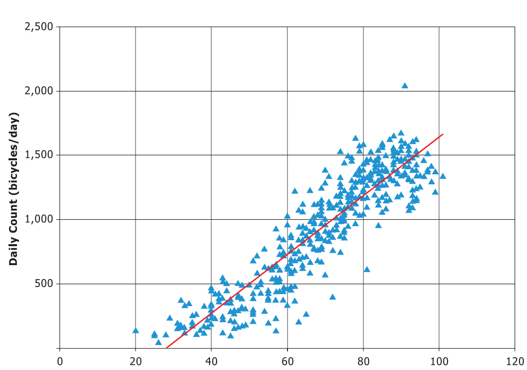


(c) Monthly Variations

Source: Municipality of Copenhagen (*12*); Portland Office of Transportation, 2007.

Exhibit 3-16
Example Variations in Bicycle Demand due to Temperature

LIVE GRAPH
Click here to view



Source: Lewin (*13*).

BICYCLE FACILITY TYPES

Exhibit 3-17 illustrates the types of bicycle facilities addressed in the HCM. The facilities are divided into two types, on-street and off-street, and include situations where a facility is shared with users of another mode (e.g., a lane shared by bicyclists and motor vehicle traffic or a pathway shared by bicyclists and pedestrians).

Exhibit 3-17
Bicycle Facility Types

(a) Shared Lane

(b) Bicycle Lane

(c) Shoulder Bikeway

(d) Sidepath

(e) Bicycle Track

(f) Exclusive Pathway

On-Street Bicycle Facilities

On-street bicycle facilities include roadways on which bicycles share a travel lane with motorized vehicular traffic, dedicated on-street bicycle lanes, paved roadway shoulders available for use by bicyclists, and bicycle tracks—mainly seen in Europe—located between the roadway and the sidewalk and separated from each by low curbs. For each facility type, bicycle flow is one-way. The quality of bicycle flow, safety, and the bicycling environment are all considerations for these types of facilities.

Off-Street Bicycle Facilities

Off-street bicycle facilities consist of pathways dedicated to the exclusive use of bicyclists and pathways shared with pedestrians and other types of users. These types of facilities may be located parallel and in proximity to roadways (*sidepaths*), or they may be completely independent facilities, such as recreational trails along former railroad rights-of-way and off-street pathways of the kind found in city parks and on college campuses. Bicycle flow along these types of facilities is typically two-way and is often shared with users of other modes. The number of meeting and passing events between cyclists and other path users affects the quality of service for bicyclists using these facility types. The presence and design of driveways and intersections may affect the quality of service of bicyclists on side paths but is not addressed by HCM procedures.

MEASURED AND OBSERVED VOLUMES

Exhibit 3-18 provides daily two-directional bicycle volumes on selected U.S. roadways and bicycle facilities. Data were collected on weekdays, generally in late spring or summer, so these volumes represent peak conditions for the year. In comparison, the highest-volume roadways in Copenhagen, Denmark, have two-directional daily bicycle volumes exceeding 30,000 on spring and fall weekdays with good weather (*12*).

Exhibit 3-18
Daily Weekday Bicycle Volumes on Selected U.S. Bicycle Facilities (2006–2008)

Facility	Location	Facility Type	Daily Bicycle Volume
Hawthorne Bridge	Portland, OR	Shared pedestrian–bicycle path	7,400
Russell Boulevard	Davis, CA	Shared pedestrian–bicycle path	6,700
Broadway Bridge	Portland, OR	Shared pedestrian–bicycle path	4,300
North Vancouver Street	Portland, OR	Bicycle lane	3,500
Hudson River Greenway	New York, NY	Shared pedestrian–bicycle path	3,500
Steel Bridge	Portland, OR	Shared pedestrian–bicycle path	3,000
Waterfront Park	Portland, OR	Shared pedestrian–bicycle path	3,000
Williamsburg Bridge	New York, NY	Shared pedestrian–bicycle path	3,000
Springwater Trail	Portland, OR	Shared pedestrian–bicycle path	2,800

Source: City of Davis, Portland Office of Transportation, New York City DOT.

INTERACTIONS WITH OTHER MODES

Automobiles

Vehicular and heavy vehicle traffic volumes and speeds, the presence of on-street parking (which presents the potential for bicyclists to hit or be hit by car doors), and the degree to which bicyclists are separated from automobile traffic all influence bicyclists' perceptions of the quality of service received while using an on-street bicycle facility. Turning vehicles, particularly right-turning vehicles that cross the path of bicyclists, also affect quality of service.

Pedestrians

The effect of pedestrians on bicycles is greatest on pathways shared by the two modes. Pedestrians—because of their markedly lower speeds and tendency to travel in groups several abreast—can have a negative effect on bicycle quality of service on such pathways. Similar to pedestrian impacts on motor vehicles, bicyclists must yield to crossing pedestrians, and the signal timing at intersections reflects, in part, the time required for pedestrians to cross the street.

Transit

Transit vehicles interact with bicycles in much the same way as automobiles. However, because transit vehicles are heavy vehicles, the effect of a single transit vehicle is proportionately greater than that of a single automobile. Buses can also affect bicyclists when they pull over into a bicycle lane or paved shoulder to serve a bus stop; however, this impact is not accounted for in HCM procedures. Although not addressed by HCM procedures, the availability of good bicycle access extends the capture shed of a transit stop or station, and when bicycles can be transported by transit vehicles, transit service can greatly extend the range of a bicycle trip.

5. TRANSIT MODE

OVERVIEW

Transit plays two major roles in North America. First, it accommodates choice riders—those who choose transit for their mode of travel even though they have other means available. These riders choose transit to avoid congestion, save money on fuel and parking, use their travel time productively for other activities, and reduce the impact of automobile driving on the environment, among other reasons. Transit is essential for mobility in the central business districts (CBDs) of some major cities, which could not survive without it.

The other major role of transit is to provide basic mobility for segments of the population that are unable to drive for age, physical, mental, or financial reasons. About 11% of the adult population in the United States does not have a driver's license (*14*) and must depend on others to transport them in automobiles, on transit, or via other modes, including walking, bicycling, and taxis. These transit users have been termed captive riders.

Exhibit 3-19 provides examples of peak-hour trips by transit to the downtowns of selected U.S. cities. The variations in transit use reflect differences in population, CBD employment, extent of bus and rail transit services, and geographic characteristics.

Exhibit 3-19
Transit Commuting to Downtowns (2000)

Area	Transit Share of Downtown Commuters	Area	Transit Share of Downtown Commuters
New York	76.5%	Los Angeles	20.3%
Chicago	61.7%	Houston	16.7%
Boston	50.9%	Dallas	13.6%
San Francisco	49.0%	Sacramento	12.3%
Philadelphia	45.7%	San Diego	11.4%
Washington, DC	37.8%	San Antonio	7.2%
Seattle	36.8%	Austin	3.8%
Portland	27.5%		

Source: *Commuting in America III* (*15*), adapted from "Commuting to Downtown in America: Census 2000," report to the Transportation Research Board Subcommittee on Census Data for Transportation Planning, Jan. 10, 2005.

HUMAN FACTORS

Transit passengers frequently rely on other modes to gain access to transit. Transit use is greater where population densities are higher and pedestrian access is good. Typical transit users do not have transit service available at the door and must walk, bicycle, or drive to a transit stop and walk or bicycle from the transit discharge point to their destination. In contrast, suburban areas are mainly automobile-oriented, with employment and residences dispersed, often without sidewalks or direct access to transit lines. If potential passengers cannot access service at both ends of their trip, transit is not an option for that trip.

Unlike the other modes addressed in the HCM, transit is primarily focused on a service rather than a facility. Roadways, bicycle lanes, and sidewalks, once constructed, are generally available at all times to users. Transit service, in contrast, is only available at designated times and places. Another important difference is that transit users are passengers, rather than drivers, and not in

direct control of their travel. Thus, the frequency and reliability of service are important quality-of-service factors for transit users. Travel speed and comfort while making a trip are also important to transit users.

Transit is about moving people rather than vehicles. Transit operations at their most efficient level involve relatively few vehicles, each carrying a large number of passengers. In contrast, roadway capacity analysis typically involves relatively large numbers of vehicles, most carrying only a single occupant. In evaluating priority measures for transit and automobile users, the number of people affected is often more relevant than the number of vehicles.

ON-STREET TRANSIT CHARACTERISTICS

The TCQSM comprehensively addresses transit modes.

The HCM addresses only those major transit modes (in terms of passengers carried) that operate on streets and interact with other users of streets and highways. These modes are buses, streetcars, and light rail, illustrated in Exhibit 3-20 and described briefly in the following sections.

Exhibit 3-20
Transit Modes Addressed in the HCM

(a) Bus (b) Streetcar (c) Light Rail

Bus

Demand-responsive transit (with flexible schedules or routes, or both) is not addressed by the HCM but is addressed in the TCQSM.

The bus mode is operated by rubber-tired vehicles that follow fixed routes and schedules along roadways. Although the electric trolleybus (a bus receiving its power from overhead wires) is classified as a separate mode by the Federal Transit Administration (FTA), for the purposes of the HCM, it is also considered a bus. In 2007, more than 53% of all transit passenger boardings in the United States occurred on buses (*16*).

The bus mode offers considerable operational flexibility. Service can range from local buses stopping every two to three blocks along a street, to limited-stop service stopping every ½ to 1 mi, to express service that travels along a roadway without stopping. Buses may stop in the travel lane (*on-line)* or in a parking lane or pullout (*off-line*). On-line stops reduce bus delay but may increase vehicle and bicycle delay. Because buses frequently carry more people than the vehicles stopped behind them, on-line stops may help reduce overall person delay.

Streetcar

The streetcar mode is operated by vehicles that receive power from overhead wires and run on tracks. For FTA reporting purposes, streetcars are considered to be a form of light rail. For HCM purposes, the following operating and physical characteristics distinguish streetcars from light rail:

- Generally shorter (typically one-car) trains, with slightly narrower cars;
- Greater potential for onboard fare payment, which may affect dwell times;

- Greater prevalence of mixed-traffic operation; and
- More frequent stops.

Streetcars make on-line stops, which minimizes their delay leaving a station, but may increase delay to vehicles and bicycles stopped behind the streetcar.

Light Rail

As is streetcar, light rail is a mode operated by vehicles that receive power from overhead wires and that run on tracks. Trains typically consist of multiple cars; fares are typically paid to a machine on the station platform (thus allowing passengers to board through all doors, reducing dwell time); station spacing tends to be relatively long, particularly outside downtown areas; and traffic signal preemption or priority is frequently employed.

The HCM only addresses light rail operations along roadways. Consult the TCQSM for a more comprehensive treatment of light rail.

When light rail operates along a roadway, it typically does so in an exclusive lane or in a segregated right-of-way in the street median or along the side of the street. Most light rail routes include lengthy sections where tracks are located in a separate, potentially grade-separated right-of-way and any interaction with traffic occurs at gate-controlled grade crossings.

TRAVEL TIME VARIABILITY

As do motorists, transit riders often allow extra time for time-sensitive trips. Sources of bus travel time variability include all of the factors listed in the automobile section of this chapter, plus bus-specific factors such as variations in passenger demands (day-to-day and bus-to-bus), traffic signal delays after stopping to serve passengers, wheelchair lift and bicycle rack usage, differences in bus operator experience, route length, and the number of stops (*9*).

ON-STREET TRANSIT FACILITY TYPES

Mixed Traffic

More than 99% of the bus route miles in the United States are operated in mixed traffic. In contrast, most rail route miles—other than portions of streetcar lines—operate in some form of segregated right-of-way. In mixed traffic, transit vehicles are subject to the same causes of delay as are automobiles, and they need to stop periodically to serve passengers. These stops can cause transit vehicles to fall out of any traffic signal progression that might be provided along the street, causing them to incur greater amounts of signal delay than other vehicles.

Exclusive Lanes

Exclusive lanes are on-street lanes dedicated for use by transit vehicles, on either a full-time or part-time basis. They are generally separated from other lanes by just a stripe, and buses may be able to leave the exclusive lane to pass buses or obstructions such as delivery trucks. Right-turning traffic, bicycles, carpools, and taxis are sometimes allowed in exclusive bus lanes. Generally, no other traffic, with the possible exception of transit buses, is allowed in exclusive lanes provided for rail transit vehicles. Exclusive lanes allow transit vehicles to bypass queues of vehicles in the general traffic lanes and reduce or eliminate

delays to transit vehicles caused by right-turning traffic. Therefore, these lanes can provide faster, more reliable transit operations.

On-Street Transitways

Buses and trains sometimes operate within a portion of the street right-of-way that is physically segregated from other traffic: in the median or adjacent to one side of the street. No other traffic is allowed in the transitway. The amount of green time allocated to transit vehicles may be different from the amount of time allocated to the parallel through movements—for example, it might be reduced to provide time to serve conflicting vehicular turning movements.

MEASURED AND OBSERVED VOLUMES

Exhibit 3-21 provides examples of observed peak-direction, peak-hour bus and passenger flows along selected roadways in the United States and Canada. Higher bus volumes have been observed on busways in South America; they require a combination of passing lanes at stations for express services and multiple-platform stations at major transfer points and downtown terminals.

Exhibit 3-21
Observed On-Street, Peak-Direction, Peak-Hour Bus and Passenger Volumes (1995–2000)

		Peak-Hour Peak-Direction	
Location	**Facility**	**Buses**	**Passengers**
Ottawa	Albert St./Slater St.	190	10,000
New York City	Madison Ave.	180	10,000
Portland	5th Ave./6th Ave.	175	8,500
Newark	Broad St.	150	6,000
Denver	Broadway/Lincoln St.	90	2,300
Boston	South St./High St.	50	2,000
Vancouver	Granville Mall	70	1,800

Source: *TCRP Report 26* (*17*), *TCRP Report 90, Volume 2* (*18*), TCQSM (*9*).

INTERACTIONS WITH OTHER MODES

Automobiles

Higher automobile volumes result in greater delays for all motorized traffic, including buses. In locations where buses pull out of the travel lane to serve bus stops and yield-to-bus laws are not in place (or generally observed), buses experience delay waiting for a gap to pull back into traffic after serving a stop. Day-to-day variations in roadway congestion and trip-to-trip variations in making or missing green phases at signalized intersections affect bus schedule reliability, although no HCM techniques exist to predict this impact.

Pedestrians

Transit users are typically pedestrians immediately before and after their trip aboard a transit vehicle, so the quality of the pedestrian environment along access routes to transit stops also affects the quality of the transit trip. Pedestrians can also delay buses in the same way that they delay automobiles, as described earlier in this chapter.

Bicycles

In locations where buses pull out of the travel lane to serve bus stops, bicycles may delay buses waiting for a gap to pull back into traffic, similar to

automobiles. Transit users may be bicyclists before or after their trip, so the quality of the bicycling environment along access routes to transit stops also influences the quality of the transit trip.

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CHAPTER 4
TRAFFIC FLOW AND CAPACITY CONCEPTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

The relationships between volume (flow rate), speed, and density are among the most fundamental in transportation engineering. **Chapter 4, Traffic Flow and Capacity Concepts,** describes how these basic relationships apply to the four modes covered by the *Highway Capacity Manual* (HCM): automobiles, pedestrians, bicycles, and on-street transit. Details of these relationships specific to automobiles operating on a particular system element (for example, speed–flow curves for freeways) are provided in the appropriate methodological chapters of Volumes 2 and 3.

Capacity represents the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions. Reasonable expectancy is the basis for defining capacity. The stated capacity for a given system element is a flow rate that can be achieved repeatedly for peak periods of sufficient demand, as opposed to being the maximum flow rate that might ever be observed. System elements that have different prevailing conditions will have different capacities, and the maximum flow rate observed on a given system element may vary from day to day.

Several of the operational performance measures presented in Chapter 4 (speed, delay, and density, in particular) are used in Chapter 5 to describe the quality of service provided by a roadway, or—in the case of the volume-to-capacity (demand-to-capacity) ratio—are used to define the threshold between Levels of Service (LOS) E and F.

2. AUTOMOBILE MODE

A few basic variables—volume, flow rate, speed, and density—can be used to describe traffic flow on any roadway. In the HCM, volume, flow rate, and speed are parameters common to both uninterrupted- and interrupted-flow facilities, but density applies primarily to uninterrupted flow. Some parameters related to flow rate, such as spacing and headway, are also used for both types of facilities. Other parameters, such as saturation flow and gap, are specific to interrupted flow.

BASIC AUTOMOBILE FLOW PARAMETERS

Volume and Flow Rate

Volume and flow rate are two measures that quantify the number of vehicles passing a point on a lane or roadway during a given time interval. These terms are defined as follows:

- *Volume*—the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval; any time interval can be used, but volumes are typically expressed in terms of annual, daily, hourly, or subhourly periods.
- *Flow rate*—the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1 h, usually 15 min. This chapter focuses on flow rate and the variations in flow that can occur over the course of an hour.

Flow rate is the equivalent hourly volume that would occur if a subhourly flow was sustained for an entire hour.

There is a distinction between volume and flow rate. Volume is the number of vehicles observed or predicted to pass a point during a time interval. Flow rate represents the number of vehicles passing a point during a time interval less than 1 h, but expressed as an equivalent hourly rate. A flow rate is the number of vehicles observed in a subhourly period, divided by the time (in hours) of the observation. For example, a volume of 100 veh observed in a 15-min period implies a flow rate of 100 veh divided by 0.25 h, or 400 veh/h.

Volume and flow rate are variables that help quantify *demand,* that is, the number of vehicle occupants or drivers (usually expressed as the number of vehicles) who desire to use a given system element during a specific time period, typically 1 h or 15 min. As discussed in Chapter 3, Modal Characteristics, observed volumes may reflect upstream capacity constraints rather than the true demand that would exist without the presence of a bottleneck.

Observed volumes may reflect capacity constraints rather than true demand. Demand is usually the desired input to HCM analyses, although it is not always easy to determine.

In many cases, demand volumes are the desired input to HCM analyses. (An exception would be, for example, when one is interested in analyzing traffic conditions downstream of a bottleneck that is not planned to be removed.) When conditions are undersaturated and no upstream bottlenecks exist, demand volume at a location can be assumed to be equivalent to the measured volume at that location. Otherwise, ascertaining demand requires a count of undersaturated traffic upstream of a bottleneck (i.e., a count of arrival volume rather than departure volume) (*1*). When the queue from a bottleneck extends past the

previous intersection or interchange, it may not be easy to determine how much of the traffic approaching the end of the queue is actually destined for the bottleneck location. Furthermore, as illustrated in Chapter 3, demand patterns may change after a bottleneck is removed. Nevertheless, where bottlenecks exist, neglecting to use demand volumes as inputs to HCM methodologies will produce results that underestimate the presence and extent of congestion. In other words, using observed volumes instead of demand volumes will likely result in inaccurate HCM results.

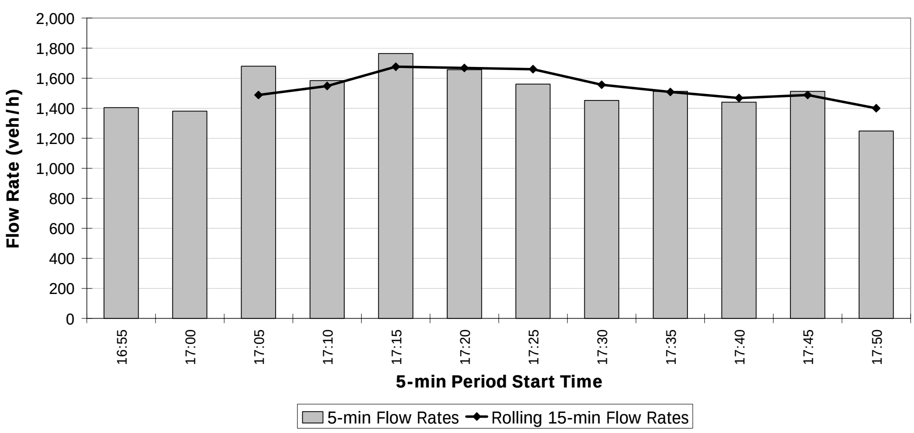
Where bottlenecks exist, not accounting for demand will result in underestimating the extent of congestion.

Subhourly Variations in Flow

Flow rates typically vary over the course of an hour. Exhibit 4-1 shows the substantial short-term fluctuation in flow rate that can occur within an hour, on the basis of data from the approaches to an all-way STOP-controlled intersection. In this data set, the 5-min flow rate ranges from a low of 1,248 veh/h to a high of 1,764 veh/h, compared with a total peak hour entering volume of 1,516 veh. Designing the intersection to accommodate the peak hour volume would result in oversaturated conditions for a substantial portion of the hour.

Even when hourly volumes are less than a system element's capacity, flow rates within an hour may exceed capacity, creating oversaturated conditions.

Exhibit 4-1
Relationship Between Short-Term and Hourly Flows



Note: SW 72nd Avenue at Dartmouth Street, Tigard, Oregon, 2008.

HCM analyses typically consider the peak 15 min of flow during the analysis hour. As illustrated in Exhibit 4-1, the use of a peak 15-min flow rate accommodates nearly all the variations in flow during the hour and therefore provides a good middle ground between designing for hourly volumes and designing for the most extreme 5-min flow rate.

Since inputs to HCM procedures are typically expressed in terms of hourly demands, the HCM uses the *peak hour factor* (PHF) to convert an hourly volume into a peak 15-min flow rate. Although traditionally called a "peak hour" factor, a PHF is applicable to any analysis hour, peak or off-peak. The PHF is the ratio of total hourly volume to the peak flow rate within the hour and is computed by Equation 4-1:

Peak hour factor (PHF) defined.

Equation 4-1

$$PHF = \frac{\text{Hourly volume}}{\text{Peak flow rate (within the hour)}}$$

If 15-min periods are used, the PHF may be computed by Equation 4-2:

Equation 4-2

$$PHF = \frac{V}{4 \times V_{15}}$$

where

PHF = peak hour factor,

V = hourly volume (veh/h), and

V_{15} = volume during the peak 15 min of the analysis hour (veh/15 min).

When the PHF is known, it can convert a peak hour volume to a peak flow rate, as in Equation 4-3:

Equation 4-3

$$v = \frac{V}{PHF}$$

where v is the flow rate for a peak 15-min period, expressed in vehicles per hour, and the other variables are as defined previously.

Equation 4-3 does not need to be used to estimate peak flow rates if traffic counts are available; however, the chosen count interval must identify the maximum 15-min flow period. Then the rate can be computed directly as 4 times the maximum 15-min count and the PHF would take the value 1.00.

Lower PHF values signify greater variability of flow, while higher values signify less flow variation within the hour. When hourly counts are used, the PHF can range from 1.00, indicating that the same demand occurs during each 15-min period of the hour, to a theoretical minimum of 0.25, indicating that the entire hourly demand occurs during the peak 15 min. PHFs in urban areas generally range between 0.80 and 0.98. PHFs over 0.95 are often indicative of high traffic volumes, sometimes with capacity constraints on flow during the peak hour. PHFs under 0.80 occur in locations with highly peaked demand, such as schools, factories with shift changes, and venues with scheduled events.

Speed

Although traffic volumes provide a method of quantifying capacity values, speed (or its reciprocal, *travel time rate*) is an important measure of the quality of the traffic service provided to the motorist. It defines LOS for two-lane highways and urban streets.

Speed parameters.

Speed is defined as a rate of motion expressed as distance per unit of time, generally as miles per hour (mi/h). To characterize the speed of a traffic stream, a representative value must be used, because a broad distribution of individual speeds is observable in the traffic stream. Several speed parameters can be applied to a traffic stream. Among them are the following:

Average travel speed.

- *Average travel speed*. A traffic stream measure based on travel time observed on a known length of highway. It is the length of the segment divided by the average travel time of vehicles traversing the segment, including all stopped delay times. It is also equal to the space mean speed.

- *Space mean speed*. A statistical term denoting an average speed based on the average travel time of vehicles to traverse a length of roadway. It is called a space mean speed because the average travel time weights the average by the time each vehicle spends in the defined roadway segment or space.
- *Time mean speed*. The arithmetic average of speeds of vehicles observed passing a point on a highway; also referred to as the *average spot speed.* The individual speeds of vehicles passing a point are recorded and averaged arithmetically.
- *Free-flow speed*. The average speed of vehicles on a given segment, measured under low-volume conditions, when drivers are free to drive at their desired speed and are not constrained by the presence of other vehicles or downstream traffic control devices (i.e., traffic signals, roundabouts, or STOP signs).
- *Average running speed*. A traffic stream measure based on the observation of travel times of vehicles traversing a section of highway of known length. It is the length of the segment divided by the average running time of vehicles that traverse the segment. Running time includes only time during which vehicles are in motion.

Space mean speed.

Time mean speed.

Free-flow speed.

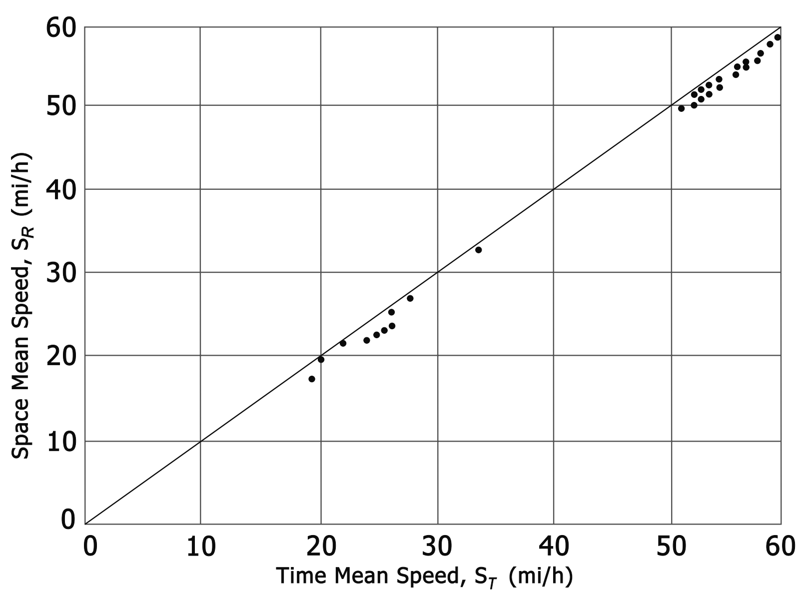
Average running speed.

For most of the HCM procedures using speed as a service measure, average travel speed (or its equivalent, space mean speed) is the defining parameter. On uninterrupted-flow facilities operating with undersaturated flow, the average travel speed is equal to the average running speed.

Exhibit 4-2 shows a typical relationship between time mean and space mean speeds. Space mean speed is always less than or equal to time mean speed, but the difference decreases as the absolute value of speed increases. Based on the statistical analysis of observed data, this relationship is useful because time mean speeds are typically easier to measure in the field than are space mean speeds.

Exhibit 4-2
Typical Relationship Between Time Mean and Space Mean Speeds

LIVE GRAPH
Click here to view



Source: Drake et al. (*2*).

It is possible to calculate both time mean speed and space mean speed from a sample of individual vehicle speeds. For example, three vehicles are recorded with speeds of 30, 40, and 50 mi/h. The times to traverse 1 mi are 2.0 min, 1.5 min, and 1.2 min, respectively. The time mean speed is 40 mi/h, calculated as (30 + 40 + 50)/3. The space mean speed is 38.3 mi/h, calculated as (60)[3/(2.0 + 1.5 + 1.2)].

Space mean speed is recommended for HCM analyses. Speeds are best measured by observing travel times over a known length of highway. For uninterrupted-flow facilities operating in the range of stable flow, the length may be as short as several hundred feet for ease of observation.

Density

Density is the number of vehicles occupying a given length of a lane or roadway at a particular instant. For the computations in this manual, density is averaged over time and is usually expressed as vehicles per mile (veh/mi) or passenger cars per mile (pc/mi).

Computing density.

Measuring density directly in the field is difficult: it requires a vantage point for photographing, videotaping, or observing significant lengths of highway. Density can be computed, however, from the average travel speed and flow rate, which are measured more easily. Equation 4-4 is used for undersaturated traffic conditions.

Equation 4-4

$$D = \frac{v}{S}$$

where

v = flow rate (veh/h),

S = average travel speed (mi/h), and

D = density (veh/mi).

A highway segment with a flow rate of 1,000 veh/h and an average travel speed of 50 mi/h would have a density of

$$D = \frac{1{,}000 \text{ veh/h}}{50 \text{ mi/h}} = 20 \text{ veh/mi}$$

Density is a critical parameter for uninterrupted-flow facilities because it characterizes the quality of traffic operations. It describes the proximity of vehicles to one another and reflects the freedom to maneuver within the traffic stream.

Roadway occupancy is frequently used as a surrogate for density in control systems because it is easier to measure (most often through equipment such as loop detectors). Occupancy in space is the proportion of roadway length covered by vehicles, and occupancy in time identifies the proportion of time a roadway cross section is occupied by vehicles. However, unless the length of vehicles is known precisely, the conversion from occupancy to density involves some error.

Headway and Spacing

Spacing is the distance between successive vehicles in a traffic stream, measured from the same point on each vehicle (e.g., front bumper, front axle). Headway is the time between successive vehicles as they pass a point on a lane or roadway, also measured from the same point on each vehicle.

These characteristics are microscopic, because they relate to individual pairs of vehicles within the traffic stream. Within any traffic stream, both the spacing and the headway of individual vehicles are distributed over a range of values, generally related to the speed of the traffic stream and prevailing conditions. In the aggregate, these microscopic parameters relate to the macroscopic flow parameters of density and flow rate.

Spacing is a distance, measured in feet. It can be determined directly by measuring the distance between common points on successive vehicles at a particular instant. This generally requires costly aerial photographic techniques, so that spacing usually derives from other direct measurements. Headway, in contrast, can be measured with stopwatch observations as vehicles pass a point on the roadway.

Relationships among density, speed and flow rate, and headway and spacing.

The average vehicle spacing in a traffic stream is directly related to the density of the traffic stream, as determined by Equation 4-5:

Equation 4-5

$$\text{Density (veh/mi)} = \frac{\text{5,280 ft/mi}}{\text{spacing (ft/veh)}}$$

The relationship between average spacing and average headway in a traffic stream depends on speed, as indicated in Equation 4-6:

Equation 4-6

$$\text{Headway (s/veh)} = \frac{\text{spacing (ft/veh)}}{\text{speed (ft/s)}}$$

This relationship also holds for individual headways and spacings between pairs of vehicles. The speed used is that of the second vehicle in a pair. The flow rate is related to the average headway of the traffic stream by Equation 4-7:

Equation 4-7

$$\text{Flow rate (veh/h)} = \frac{\text{3,600 s/h}}{\text{headway (s/veh)}}$$

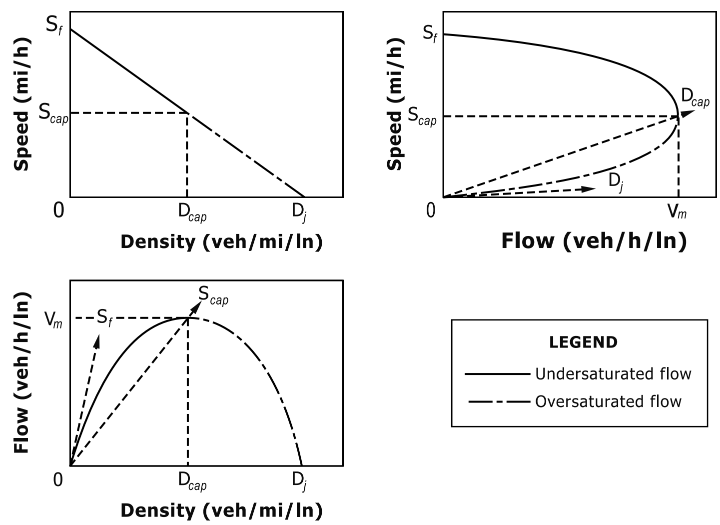
Relationships Among Basic Parameters

Equation 4-4 cites the basic relationship among the three parameters, describing an uninterrupted traffic stream. Although the equation $v = S \times D$ algebraically allows for a given flow rate to occur in an infinite number of combinations of speed and density, there are additional relationships that restrict the variety of flow conditions that can occur at a location.

Exhibit 4-3 shows a generalized, theoretical representation of these relationships, which are the basis for the capacity analysis of uninterrupted-flow facilities. The flow–density function is placed directly below the speed–density relationship because of their common horizontal scales, and the speed–flow function is placed next to the speed–density relationship because of their common vertical scales. The speed in all cases is *space mean speed*.

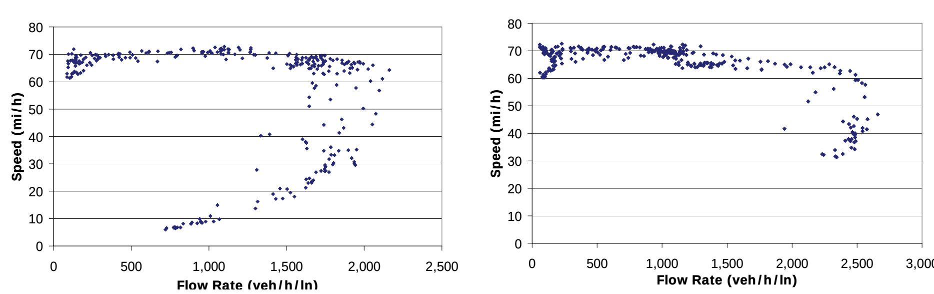
The form of these functions depends on the prevailing traffic and roadway conditions on the segment under study and on the segment length. Although the diagrams in Exhibit 4-3 show continuous curves, it is unlikely that the full range of the functions would appear at any particular location. Real-world data usually show discontinuities, with parts of the curves not present (*3*). Exhibit 4-4 shows that the real-world relationship between speed and undersaturated flow consists of a section of constant speed, followed by a section of declining speed until capacity is reached, unlike the idealized parabola shown in the speed–flow curve in Exhibit 4-3. Exhibit 4-4(a) shows a relatively complete curve, while Exhibit 4-4(b) has discontinuities.

Exhibit 4-3
Generalized Relationships Among Speed, Density, and Flow Rate on Uninterrupted-Flow Facilities



Source: Adapted from May (*3*).

Exhibit 4-4
Example Freeway Speed–Flow Data



(a) I-405, Los Angeles, California

(b) I-805, San Diego, California

Source: California Department of Transportation, 2008.

Note that the real-world speed–flow curves in Exhibit 4-4 are not the idealized parabola indicated in Exhibit 4-3. The other relationships in Exhibit 4-3 therefore also have somewhat different shapes in the real world.

LIVE GRAPH
Click here to view

LIVE GRAPH
Click here to view

The curves of Exhibit 4-3 illustrate several significant points. A zero flow rate occurs under two different conditions. The first is when there are no vehicles on the segment—density is zero, and flow rate is zero. Speed is theoretical for this condition and would be selected by the first driver (presumably at a high value). This speed is represented by S_f in the graphs.

The second is when density becomes so high that all vehicles must stop—the speed and flow rate are zero because there is no movement and vehicles cannot pass a point on the roadway. The density at which all movement stops is called *jam density*, denoted by D_j in the diagrams.

Between these two extreme points, the dynamics of traffic flow produce a maximizing effect. As flow increases from zero, density also increases because more vehicles are on the roadway. When this happens, speed declines because of the interaction of vehicles. The decline is negligible at low and medium densities and flow rates. As density increases, the generalized curves suggest that speed decreases significantly before capacity is achieved. Capacity is reached when the product of density and speed results in the maximum flow rate. This condition is shown as the speed at capacity S_{cap} (often called *critical speed*), density at capacity D_{cap} (sometimes referred to as *critical density*), and maximum flow v_m.

The slope of any ray drawn from the origin of the speed–flow curve represents the inverse of density, on the basis of Equation 4-4. Similarly, a ray in the flow–density graph represents speed. As examples, Exhibit 4-3 shows the average free-flow speed and speed at capacity, as well as optimum and jam densities. The three diagrams are redundant—if any one relationship is known, the other two are uniquely defined. The speed–density function is used mostly for theoretical work; the other two are used in this manual to define LOS for freeways and multilane highways.

Exhibit 4-3 shows that any flow rate other than capacity can occur under two conditions, one low density and high speed and the other high density and low speed. The high-density, low-speed side of the curves represents oversaturated flow. Sudden changes can occur in the state of traffic (i.e., in speed, density, and flow rate). LOS A though E are defined on the low-density, high-speed side of the curves, with the maximum-flow boundary of LOS E placed at capacity; in contrast, LOS F, which describes oversaturated and queue discharge traffic, is represented by the high-density, low-speed part of the curves.

ADDITIONAL UNINTERRUPTED-FLOW PARAMETERS

The average headway in a lane is the reciprocal of the flow rate. Thus, at a flow of 2,400 veh/h/ln, the average headway is 3,600 s/h divided by 2,400 veh/h, or 1.5 s/veh. Vehicles do not, however, travel at constant headways. Vehicles tend to travel in groups, or platoons, with varying headways between successive vehicles.

An example of the distribution of headways observed on the Long Island Expressway is shown in Exhibit 4-5. The headway distribution of Lane 3 is the most nearly uniform, as evidenced by the range of values and the high frequency of the modal value, which is the peak of the distribution curve. The distribution of Lane 2 is similar to that of Lane 3, with slightly greater scatter (range from 0.5 to 9.0 s). Lane 1 shows a much different pattern: it is more dispersed, with headways ranging from 0.5 to 12.0 s, and the frequency of the modal value is only about one-third of that for the other lanes. This indicates that the flow rate in the shoulder lane is usually lower than the flow rates in the adjacent lanes when the total flows on this segment are moderate to high.

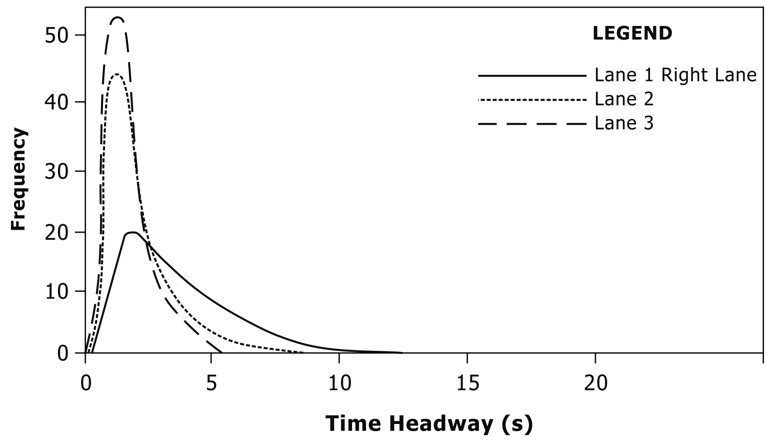
__Headway__ includes the vehicle length, while __gap__ is the space between vehicles.

Exhibit 4-5 shows relatively few headways smaller than 1.0 s. A vehicle traveling at 60 mi/h (88 ft/s) would have a spacing of 88 ft with a 1.0-s headway, and only 44 ft with a 0.5-s headway. This effectively reduces the space between

vehicles (rear bumper to front bumper) to only 25 to 30 ft. This spacing (also called *gap*) would be extremely difficult to maintain.

Exhibit 4-5
Time Headway Distribution for Long Island Expressway

LIVE GRAPH
Click here to view



Source: Berry and Gandhi (*4*).

Drivers react to this intervehicle spacing, which they perceive directly, rather than to headway. Headway includes the length of the vehicle, which became smaller for passenger cars in the vehicle mix of the 1980s. In the 1990s and 2000s, because of the popularity of sport utility vehicles, typical vehicle lengths increased. If drivers maintain the same intervehicle spacing and car lengths continue to increase, conceivably, some decreases in capacity could result.

If traffic flow were truly random, small headways (less than 1.0 s) could theoretically occur. Several mathematical models have been developed that recognize the absence of small headways in most traffic streams (*5*).

ADDITIONAL INTERRUPTED-FLOW PARAMETERS

Basic concepts for interrupted-flow facilities: intersection control, saturation flow rate, lost time, and queuing.

Interrupted flow can be more complex to analyze than uninterrupted flow because of the time dimension involved in allocating space to conflicting traffic streams. On an interrupted-flow facility, flow usually is dominated by points of fixed operation, such as traffic signals and STOP signs. These controls have different impacts on overall flow.

The operational state of traffic on an interrupted-flow facility is defined by the following measures:

- Volume and flow rate (discussed earlier in the chapter),
- Saturation flow and departure headways,
- Control variables (STOP or signal control),
- Gaps available in the conflicting traffic streams, and
- Control delay.

Signalized Intersection Flow

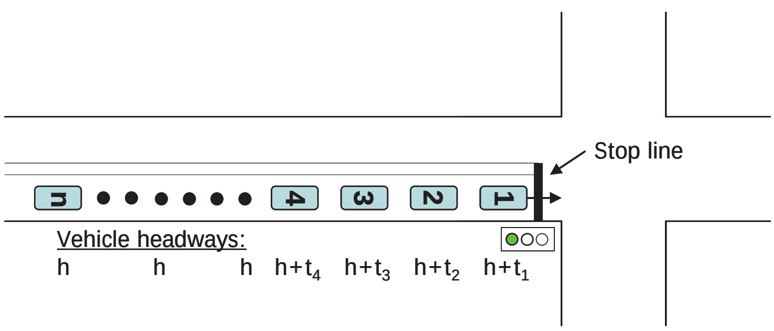
The most significant source of fixed interruptions on an interrupted-flow facility is traffic signals. A traffic signal periodically halts flow for each movement or set of movements. Movement on a given set of lanes is possible only for a portion of the total time, because the signal prohibits movement

Impact of traffic signal control on maximum flow rate.

during some periods. Only the time during which the signal is effectively green is available for movement. For example, if one set of lanes at a signalized intersection receives a 30-s effective green time out of a 90-s total cycle, only 30/90 or 1/3 of total time is available for movement on the subject lanes. Thus, flow on the lanes can occur only for 20 min of each hour. If the lanes can accommodate a maximum flow rate of 1,500 veh/h with the signal green for a full hour, they can actually accommodate a total rate of flow of only 500 veh/h, since only one-third of each hour is available as green.

When the signal turns green, the dynamics of starting a stopped queue of vehicles must be considered. Exhibit 4-6 shows a queue of vehicles stopped at a signal. When the signal turns green, the queue begins to move. The headway between vehicles can be observed as the vehicles cross the stop line of the intersection. The first headway would be the elapsed time, in seconds, between the initiation of the green and the front wheels of the first vehicle crossing over the stop line. The second headway would be the elapsed time between the front bumpers (or wheels) of the first and second vehicles crossing over the stop line. Subsequent headways are measured similarly.

Exhibit 4-6
Acceleration Headways at a Signalized Intersection

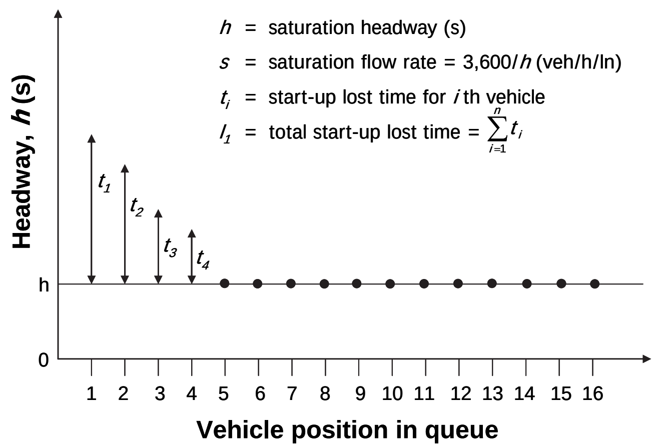


The driver of the first vehicle in the queue must observe the signal change to green and react to the change by releasing the brake and accelerating through the intersection. As a result, the first headway will be comparatively long. The second vehicle in the queue follows a similar process, except that the reaction and acceleration period can occur while the first vehicle is beginning to move. The second vehicle will be moving faster than the first as it crosses the stop line, because it has a greater distance over which to accelerate. Its headway will generally be less than that of the first vehicle. The third and fourth vehicles follow a similar procedure, each achieving a slightly lower headway than the preceding vehicle. After four vehicles, the effect of the start-up reaction and acceleration has typically dissipated. Successive vehicles then move past the stop line at a more constant headway until the last vehicle in the original queue has passed the stop line.

In Exhibit 4-6, this constant average headway, denoted as h, is achieved after four vehicles. The acceleration headways for the first four vehicles are, on the average, greater than h and are expressed as $h + t_i$, where t_i is the incremental headway for the ith vehicle due to the start-up reaction and acceleration. As i increases from 1 to 4, t_i decreases.

Exhibit 4-7 shows a conceptual plot of headways. The HCM recommends using the fifth vehicle following the beginning of a green as the starting point for saturation flow measurements.

Exhibit 4-7
Concept of Saturation Flow Rate and Lost Time



The value *h* represents the *saturation headway,* estimated as the constant average headway between vehicles after the fourth vehicle in the queue and continuing until the last vehicle that was in the queue at the beginning of the green has cleared the intersection.

Saturation flow rate.

The reference point on the vehicle used to measure headways is typically the front bumper, although front axles are sometimes used in studies that utilize tube counters to obtain the data.

Saturation flow rate is defined as the flow rate per lane at which vehicles can pass through a signalized intersection. It is computed by Equation 4-8:

Equation 4-8

$$s = \frac{3{,}600}{h}$$

where

s = saturation flow rate (veh/h), and

h = saturation headway (s).

The saturation flow rate represents the number of vehicles per hour per lane that could pass through a signalized intersection if a green signal was displayed for the full hour, the flow of vehicles never stopped, and there were no large headways. Chapter 24, Concepts: Supplemental, includes a summary of various studies of saturation flow at signalized intersections over a 25-year period.

Total start-up lost time.

Each time a flow is stopped, it must start again, with the first four vehicles experiencing the start-up reaction and acceleration headways shown in Exhibit 4-6. In this exhibit, the first four vehicles in the queue encounter headways longer than the saturation headway, *h*. The increments, t_i, are called start-up lost times. The *total start-up lost time* for the vehicles is the sum of the increments, as computed by using Equation 4-9.

$$l_1 = \sum_{i=1}^{n} t_i$$

Equation 4-9

where

l_1 = total start-up lost time (s),

t_i = lost time for ith vehicle in queue (s), and

n = last vehicle in queue.

Clearance lost time.

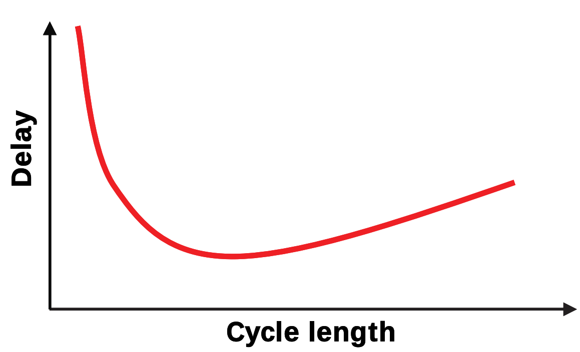
Each stop of a stream of vehicles is another source of lost time. When one stream of vehicles stops, safety requires some clearance time before a conflicting stream of traffic is allowed to enter the intersection. The interval when no vehicles use the intersection is called *clearance lost time*, l_2. In practice, signal cycles provide for this clearance through change intervals, which can include yellow or red-clearance indications, or both. Drivers generally cannot observe this entire interval but use the intersection during some portion of it.

The relationship between saturation flow rate and lost times is critical. For any given lane or movement, vehicles use the intersection at the saturation flow rate for a period equal to the available green time plus the change interval minus the start-up and clearance lost times. Because lost time is experienced with each start and stop of a movement, the total amount of time lost over an hour is related to the signal timing. For instance, if a signal has a 60-s cycle length, it will start and stop each movement 60 times per hour, and the total lost time per movement will be $60(l_1 + l_2)$.

Lost time affects capacity and delay. As indicated by the relationship of cycle length to lost time, the capacity of an intersection increases as cycle length increases. However, the capacity increase can be offset somewhat by the observation that the saturation headway, h, can increase if the length of a continuous green indication increases. Capacity increases due to longer cycles are also often offset by the increase in delay that typically results from longer cycles, as discussed below. Other intersection features, such as turning lanes, can also offset the reduced capacity that results from short cycles. Longer cycles increase the number of vehicles in the queues and can cause the left-turn lane to overflow, reducing capacity by blocking the through lanes.

As indicated in Exhibit 4-8, there is a strong relationship between delay and cycle length. For every intersection there is a small range of cycle lengths that will result in the lowest average delay for motorists. Delay, however, is a complex variable affected by many variables besides cycle length.

Exhibit 4-8
Generalized Cycle Length and Delay Relationship



STOP- and YIELD-Controlled Intersection Flow

The driver on the minor street or the driver turning left from the major street at a two-way STOP-controlled intersection faces a specific task: selecting a gap in traffic through which to execute the desired movement. The term *gap* refers to the time interval (*time gap*) and corresponding distance for a given speed (*space gap*) between the major-street vehicles entering an unsignalized intersection, measured from back bumper to front bumper. The term *gap acceptance* describes the completion of a vehicle's movement into a gap.

The capacity of a minor street approach depends on two factors:

- The distribution of available gaps in the major-street traffic stream, and
- The gap sizes required by drivers in other traffic streams to execute their desired movements.

Gap acceptance.

The distribution of available gaps in the major-street traffic stream depends on the total volume on the street, its directional distribution, the number of lanes on the major street, and the degree and type of platooning in the traffic stream. The gap sizes required by the minor-movement drivers depend on the type of maneuver (left, through, right), the number of lanes on the major street, the speed of major-street traffic, sight distances, the length of time the minor-movement vehicle has been waiting, and driver characteristics (eyesight, reaction time, age, etc.).

Critical headway.

For ease of data collection, headways (e.g., front bumper to front bumper) are usually measured instead of gaps, since only half as much data are required (i.e., only front bumper positions need to be recorded, rather than both front and back bumper positions). The *critical headway* is the minimum time interval between the front bumpers of two successive vehicles in the major traffic stream that will allow the entry of one minor-street vehicle. When more than one minor-street vehicle uses one major-street gap, the time headway between the two minor-street vehicles is called *follow-up headway.* In general, the follow-up headway is shorter than the critical headway.

The operation of roundabouts is similar to that of two-way STOP-controlled intersections. In roundabouts, however, entering drivers scan only one stream of traffic—the circulating stream—for an acceptable gap.

At an all-way STOP-controlled intersection, all drivers must come to a complete stop. The decision to proceed is based in part on the rules of the road, which suggest that the driver on the right has the right-of-way, but it is also a function of the traffic condition on the other approaches. The departure headway for the subject approach is defined as the time between the departure of one vehicle and that of the next behind it. A departure headway is considered a saturation headway if the second vehicle stops behind the first at the stop line. If there is traffic on one approach only, vehicles can depart as rapidly as the drivers can safely accelerate into and clear the intersection. If traffic is present on other approaches, the saturation headway on the subject approach will increase, depending on the degree of conflict between vehicles.

Delay

Delay is an important performance measure for interrupted-flow system elements. There are several types of delay, but *control delay*—the delay brought about by the presence of a traffic control device—is the principal service measure in the HCM for evaluating LOS at signalized and unsignalized intersections. Control delay includes delay associated with vehicles slowing in advance of an intersection, the time spent stopped on an intersection approach, the time spent as vehicles move up in the queue, and the time needed for vehicles to accelerate to their desired speed.

Control delay.

Other types of delay sometimes used are the following:

Other types of delay.

- *Geometric delay*. Delay caused by geometric features causing vehicles to reduce their speed in negotiating a system element (e.g., delay experienced where an arterial street makes a sharp turn, causing vehicles to slow, or the delay caused by the indirect route that through vehicles must take through a roundabout).
- *Incident delay*. The additional travel time experienced as a result of an incident, compared with the no-incident condition.
- *Traffic delay*. Delay resulting from the interaction of vehicles, causing drivers to reduce their speed below the free-flow speed.
- *Total delay*. The sum of control, geometric, incident, and traffic delay.

Number of Stops

Traffic control devices separate vehicles on conflicting paths by requiring one vehicle to stop or yield to the other. The stop causes delay and has an associated cost in terms of fuel consumption and wear on the vehicle. For this reason, information about stops incurred is useful in evaluating performance and calculating road user costs. This measure is typically expressed in terms of *stop rate,* which represents the count of stops divided by the number of vehicles served. Stop rate has units of stops per vehicle.

Stops are generally expected by motorists arriving at an intersection as a minor movement (e.g., a turn movement or a through movement on the minor street). However, through drivers do not expect to stop when they travel along a major street. Their expectation is that the signals will be coordinated to some degree such that they can arrive at each signal in succession while it is displaying a green indication for the through movement. For this reason, stop rate is a useful performance measure for evaluating coordinated signal systems.

Queuing

When demand exceeds capacity for a period of time or when an arrival headway is less than the service time (at the microscopic level) at a specific location, a queue forms (*3*). Queuing is both an important operational measure and a design consideration for an intersection and its vicinity. Queues that are longer than the available storage length can create several types of operational problems. A through-lane queue that extends past the entrance to a turn lane blocks access to the turn lane, keeping it from being used effectively. Similarly, a turn-lane queue overflow into a through lane interferes with the movement of

through vehicles. Queues that extend upstream from an intersection can block access into and out of driveways and—in a worst case—can spill back into and block upstream intersections, causing side streets to begin to queue back.

Several queuing measures can be calculated, including the average queue length, the maximum back of queue, and the maximum probable queue (e.g., a 95th percentile queue).

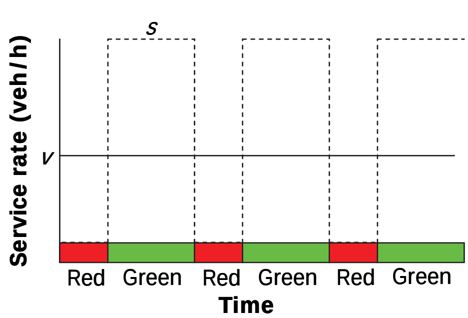
To predict the characteristics of a queuing system mathematically, it is necessary to specify the following system characteristics and parameters (*4*):

- Arrival pattern characteristics, including the average rate of arrival and the statistical distribution of time between arrivals;
- Service facility characteristics, including service-time average rates and the distribution and number of customers that can be served simultaneously or the number of channels available; and
- Queue discipline characteristics, such as the means of selecting which customer is next.

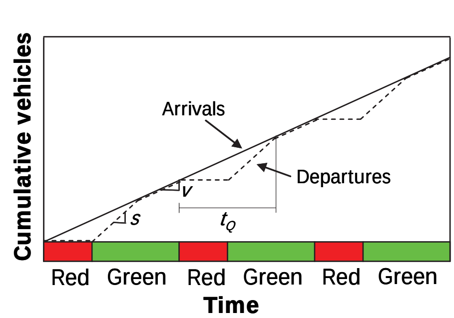
In oversaturated queues, the arrival rate is higher than the service rate; in undersaturated queues, the arrival rate is less than the service rate. The length of an undersaturated queue can vary but will reach a steady state with the arrival of vehicles. In contrast, the length of an oversaturated queue will never reach a steady state; it will increase with the arrival of vehicles until the arrival demand decreases.

An idealized undersaturated queue at a signalized intersection is shown in Exhibit 4-9 (*3*). The exhibit assumes queuing on one approach at an intersection with two signal phases. In each cycle, the arrival demand (assumed to be constant in this ideal example) is less than the capacity of the approach, no vehicles wait longer than one cycle, and there is no overflow from one cycle to the next. Exhibit 4-9(a) specifies the arrival rate, v, in vehicles per hour and is constant for the study period. The service rate, s, has two states: zero when the signal is effectively red, and up to saturation flow rate when the signal is effectively green. Note that the service rate is equal to the saturation flow rate only when there is a queue.

Exhibit 4-9
Idealized Queuing Diagram for a Two-Phase Signalized Intersection



(a) Arrival and Service Rates



(b) Queue Lengths

Exhibit 4-9(b) diagrams cumulative vehicles over time. The horizontal line, v, in Exhibit 4-9(a) becomes a solid sloping line in Exhibit 4-9(b), with the slope equal to the flow rate. Thus, the arrival rate goes through the origin and slopes up to the right with a slope equal to the arrival rate. Transferring the service rate from Exhibit 4-9(a) to Exhibit 4-9(b) creates a different graph. During the red period, the service rate is zero, so the service is shown as a horizontal dashed line in Exhibit 4-9(b). At the start of the green period, a queue is present, and the service rate is equal to the saturation flow rate. This forms a series of triangles, with the cumulative arrival line as the top side of each triangle and the cumulative service line forming the other two sides.

Each triangle represents one cycle length and can be analyzed to calculate the time duration of the queue. It starts at the beginning of the red period and continues until the queue dissipates. Its value varies between the effective red time and the cycle length, and it is computed by using Equation 4-10:

$$vt_Q = s(t_Q - r) \quad \text{or} \quad t_Q = \frac{sr}{s - v}$$

Equation 4-10

where

t_Q = time duration of queue (s),

v = mean arrival rate (veh/h),

s = mean service rate (veh/h), and

r = effective red time (s).

The queue length (i.e., the number of vehicles in the queue, as opposed to the location of the back of the queue) is represented by the vertical distance through the triangle. At the beginning of red, the queue length is zero. It increases to its maximum value at the end of the red period. Then the queue length decreases until the arrival line intersects the service line and the queue length equals zero.

The queuing characteristics can be modeled by varying the arrival rate, the service rate, and the timing plan. In real-life situations, arrival rates and service rates are continuously changing. These variations complicate the model, but the basic relationships do not change.

CAPACITY CONCEPTS

Definition of Capacity

The *capacity* of a system element is the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions.

Vehicle capacity is the maximum number of vehicles that can pass a given point during a specified period under prevailing roadway, traffic, and control conditions. This assumes that there is no influence from downstream traffic operation, such as queues backing into the analysis point.

Person capacity is the maximum number of persons that can pass a given point during a specified period under prevailing conditions. Person capacity is

commonly used to evaluate public transit services, high-occupancy-vehicle lanes, and pedestrian facilities.

Prevailing roadway, traffic, and control conditions define capacity; these conditions should be reasonably uniform for any segment of a facility that is analyzed. Any change in the prevailing conditions changes a facility's capacity.

Capacity analyses examine roadway elements under uniform traffic, roadway, and control conditions. These conditions determine capacity; therefore, segments with different prevailing conditions will have different capacities.

Capacity is defined on the basis of reasonable expectancy.

Reasonable expectancy is the basis for defining capacity. That is, the stated capacity for a given system element is a flow rate that can be achieved repeatedly for peak periods of sufficient demand. Stated capacity values can be achieved on system elements with similar characteristics throughout North America. Capacity is not the absolute maximum flow rate observed on such a system element. The absolute maximum flow rate can vary from day to day and from location to location.

Persons per hour, passenger cars per hour, and vehicles per hour are measures that can define capacity, depending on the type of system element and the type of analysis. The concept of person flow is important in making strategic decisions about transportation modes in heavily traveled corridors and in defining the role of transit and high-occupancy-vehicle priority treatments. Person capacity and person flow weight each type of vehicle in the traffic stream by the number of occupants carried.

Base Conditions

Base conditions defined.

Many of the procedures in this manual provide a formula or simple tabular or graphic presentations for a set of specified standard conditions, which must be adjusted to account for prevailing conditions that do not match. These standard conditions are termed *base conditions.*

Base conditions assume good weather, good and dry pavement conditions, users who are familiar with the system element, and no impediments to traffic flow. Other more specific base conditions are identified in each methodological chapter in Volumes 2 and 3.

Prevailing conditions almost always differ from the base conditions.

In most capacity analyses, prevailing conditions differ from the base conditions (e.g., there are trucks in the traffic stream, lanes are narrow). As a result, computations of capacity, service flow rate, and LOS must include adjustments. Prevailing conditions are generally categorized as roadway, traffic, or control.

Roadway Conditions

Impact of roadway conditions.

Roadway conditions include geometric and other elements. In some cases, they influence the capacity of a system element; in others, they can affect a performance measure such as speed, but not the roadway's capacity or maximum flow rate.

Roadway factors include the following:

- Number of lanes,
- The type of system element and its development environment,
- Lane widths,
- Shoulder widths and lateral clearances,
- Design speed,
- Horizontal and vertical alignments, and
- Availability of exclusive turn lanes at intersections.

The horizontal and vertical alignments of a highway depend on the design speed and the topography of the land on which it is constructed.

In general, as the severity of the terrain increases, capacity and service flow rates are reduced. This is significant for two-lane rural highways, where the severity of terrain can affect the operating capabilities of individual vehicles in the traffic stream and restrict opportunities for passing slow-moving vehicles.

Traffic Conditions

Traffic conditions that influence capacities and service levels include vehicle type and lane or directional distribution.

Vehicle Type

The entry of heavy vehicles—that is, vehicles other than passenger cars (a category that includes small trucks and vans)—into the traffic stream affects the number of vehicles that can be served. Heavy vehicles are vehicles that have more than four tires touching the pavement.

Trucks, buses, and recreational vehicles (RVs) are the three groups of heavy vehicles addressed by the methods in this manual. Heavy vehicles adversely affect traffic in two ways:

- They are larger than passenger cars and occupy more roadway space; and
- They have poorer operating capabilities than passenger cars, particularly with respect to acceleration, deceleration, and the ability to maintain speed on upgrades.

The second impact is more critical. The inability of heavy vehicles to keep pace with passenger cars in many situations creates large gaps in the traffic stream, which are difficult to fill by passing maneuvers. Queues may also develop behind a slow-moving heavy vehicle. The resulting inefficiencies in the use of roadway space cannot be completely overcome. This effect is particularly harmful on sustained, steep upgrades, where the difference in operating capabilities is most pronounced, and on two-lane highways, where passing requires use of the opposing travel lane.

Heavy vehicles also can affect downgrade operations, particularly when downgrades are steep enough to require operation in a low gear. In these cases, heavy vehicles must operate at speeds slower than passenger cars, again forming gaps ahead and queues behind in the traffic stream.

Trucks cover a wide range of vehicles, from lightly loaded vans and panel trucks to the most heavily loaded coal, timber, and gravel haulers. An individual truck's operational characteristics vary on the basis of the weight of its load and its engine performance.

RVs also include a broad range: campers (both self-propelled and towed), motor homes, and passenger cars or small trucks towing a variety of recreational equipment such as boats, snowmobiles, and motorcycle trailers. Although these vehicles might operate considerably better than trucks, their drivers are not professionals, which accentuates the negative impact of RVs on the traffic stream.

Intercity buses are relatively uniform in performance. Generally, urban transit buses are not as powerful as intercity buses; however, their most severe impact on traffic results from the discharge and pickup of passengers on the roadway. For the methods in this manual, the performance characteristics of buses are considered to be similar to those of trucks.

Directional and Lane Distribution

Two traffic characteristics in addition to the distribution of vehicle types affect capacity, service flow rates, and LOS: directional distribution and lane distribution. Directional distribution has a dramatic impact on two-lane rural highway operation, where optimal conditions are achieved when the amount of traffic is roughly equal in each direction. Capacity analyses for multilane highways focus on a single direction of flow. Nevertheless, each direction of the highway is usually designed to accommodate the peak flow rate in the peak direction. Typically, morning peak traffic occurs in one direction and evening peak traffic occurs in the opposite direction. Lane distribution is another factor on multilane facilities. Typically, the right lane carries less traffic than other lanes.

Driver Population

Studies have noted that noncommuter driver populations do not display the same characteristics as do regular commuters. For example, for recreational traffic on a freeway segment, capacities have been observed to be as much as 10% to 15% lower than for commuter traffic traveling on the same segment. HCM methods include an adjustment for driver population, for system elements where driver population has made a difference in the observed capacity.

Control Conditions

For interrupted-flow facilities, the control of the time that specific traffic flows are allowed to move is critical to capacity, service flow rates, and level of service. The most critical type of control is the traffic signal. The type of control in use, signal phasing, allocation of green time, cycle length, and the relationship with adjacent control measures all affect operations.

STOP and YIELD signs also affect capacity, but in a less deterministic way. A traffic signal designates times when each movement is permitted; however, a STOP sign at a two-way STOP-controlled intersection only designates the right-of-way to the major street. Motorists traveling on the minor street must stop to find gaps in the major traffic flow. The capacity of minor approaches, therefore,

depends on traffic conditions on the major street. An all-way STOP control requires drivers to stop and enter the intersection in rotation. Capacity and operational characteristics can vary widely, depending on the traffic demands on the various approaches.

Other types of controls and regulations can significantly affect capacity, service flow rates, and LOS. Restricted curb parking can increase the number of lanes available on a street or highway. Turn restrictions can eliminate conflicts at intersections, increasing capacity. Lane use controls can allocate roadway space to component movements and can create reversible lanes. One-way street routings can eliminate conflicts between left turns and opposing traffic.

Technology

Intelligent transportation systems.

Intelligent transportation systems (ITS) strategies aim to increase the safety and performance of roadway facilities. For this discussion, ITS includes any technology that allows drivers and traffic control system operators to gather and use real-time information to improve vehicle navigation, roadway system control, or both. Research on ITS has grown significantly but cannot be considered comprehensive in terms of evaluating ITS impacts on roadway capacity and quality of service.

Arterial ITS strategies that have been shown to improve vehicular throughput or reduce vehicular delay are adaptive signal control and traffic signal interconnection. A freeway ITS strategy, ramp metering, has demonstrated improved mainline throughput and speed, while incident management techniques have reduced the time required to identify and clear incidents, thus minimizing the time during which capacity is reduced as well as the associated delay. Variable freeway speed limits, combined with automated speed limit enforcement, also show promise but require additional study (*6*).

Other ITS strategies seek to shift demand to alternate routes or times, thus making better use of available system capacity and reducing delay on individual facilities. Techniques include parking availability signs at the entrances to downtown areas, value pricing, variable message signs, highway advisory radio, real-time travel time and incident information provided to computers and mobile phones, and real-time in-vehicle navigation systems (*6*).

Specific impacts of ITS strategies on roadway capacity and performance are discussed in Chapter 35, Active Traffic Management, where research is available to document those impacts.

ESTIMATION OF TRAFFIC FLOW PARAMETERS

Analyzing a roadway's performance involves assigning estimated values to traffic flow parameters as a function of either time or distance. There are three common approaches to estimating traffic flow parameters:

1. Deterministic models, such as those presented in the HCM;
2. Simulation models, which take a microscopic and stochastic approach to the representation of traffic flow; and

3. Field data observations, which attempt to measure the parameters directly by data collection and analysis.

All of these approaches can only produce estimates of the parameters of interest. Each approach involves assumptions and approximations. The three approaches are bound together by the common goal of representing field conditions accurately.

On the surface, it appears that field observations should produce the most accurate representation of traffic flow. It is difficult, however, to produce quantitative observations of some traffic phenomena in a consistent manner that avoids subjective interpretation. There are limits to the accuracy of human observation, and instrumentation of traffic flow data collection is not practical for routine field studies, except for very simple parameters such as flow rate. Field data observations require a level of effort that often exceeds the available resources. Modeling techniques have therefore been introduced as a practical, but also approximate, method of estimating required parameters. It is important that modeling techniques be based on definitions and computations that are as consistent as possible with field observations and with each other.

Vehicle trajectories are the lowest common denominator for estimating traffic flow parameters.

Vehicle trajectories have come to be recognized in the literature as the "lowest common denominator" for this purpose (*7*). Vehicle trajectories represent the "ground truth" that all measurement and analysis techniques attempt to represent. Microscopic simulation models create trajectories explicitly through algorithms that apply principles of traffic flow theory to the propagation of vehicles along a highway segment. Macroscopic deterministic models do not deal with trajectories at the same level of detail, but they attempt to produce an approximation of the results that would be obtained from trajectory analyses.

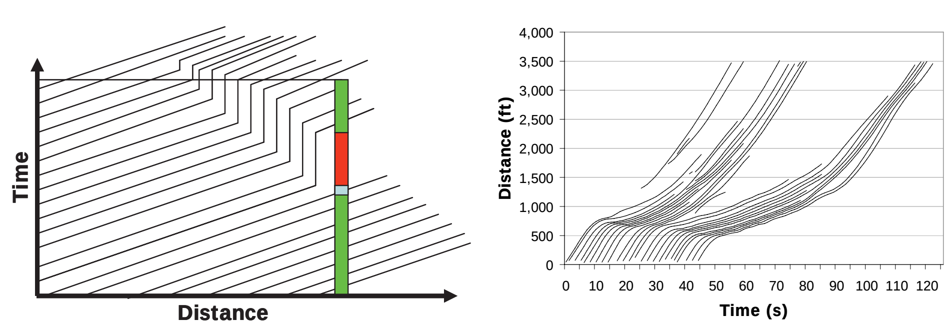
Field observations typically establish critical points along individual trajectories rather than complete trajectories.

With a few exceptions involving a significant research effort, field observations are not able to create complete trajectories. Instead, they attempt to establish critical points along individual trajectories. Because of its ability to create complete trajectories, simulation modeling may be viewed as a surrogate for field data collection through which the critical points on the trajectory may be established. It is important for this purpose that the critical points be defined in a manner that promotes compatibility between the analysis techniques.

Vehicle trajectories may be represented graphically or mathematically. The graphical representation shows the position of each vehicle in time and space as it traverses a length of the highway. Typical examples of vehicle trajectory plots are shown in Exhibit 4-10.

Exhibit 4-10
Typical Examples of Vehicle Trajectory Plots

LIVE GRAPH
Click here to view



(a) Interrupted Flow on a Signalized Approach

(b) Uninterrupted Flow on a Freeway

Exhibit 4-10(a) depicts a classical queue accumulation and release at a signalized stop line. Exhibit 4-10(b) shows a typical freeway situation in which queuing and shock waves are caused entirely by vehicle interactions and not by traffic control devices.

There are three characteristics shown in Exhibit 4-10 that are not necessarily common to all time–space representations of vehicle trajectories:

1. Time may be shown on either the vertical or horizontal axis. Note that Exhibit 4-10(a) shows time on the vertical axis, while Exhibit 4-10(b) shows time on the horizontal axis.
2. The angular shape of the interrupted-flow trajectory curves does not represent the acceleration and deceleration in their true forms. This shape displays an approximation of the trajectory that is appropriate for some interpretations and inappropriate for others.
3. Both plots represent a single lane of operation in which each vehicle follows its leader according to established rules. Multiple-lane trajectory plots differ from single-lane plots in two ways. First, the first in, first out queue discipline can be violated in multilane situations because of overtaking. In other words, a vehicle entering a link later than its leader could leave the link earlier. Graphically, this situation is represented by trajectory lines crossing each other. Second, some vehicles might change lanes. Lane changes cannot be represented in the Exhibit 4-10 plots because distance is shown as a one-dimensional scalar quantity. Because of these complexities, multiple-lane trajectories are much harder to analyze.

While the two plots shown in Exhibit 4-10 provide good visual insights into vehicle operations, they do not support any quantitative assessments. To develop performance measures from vehicle trajectories, it is necessary to represent them mathematically, rather than visually. A mathematical representation requires the development of a set of properties that are associated with each vehicle at specific points in time and space. Because of the time-step formulation of most simulation models, it is preferable to choose time as the reference point instead of distance.

The key to producing performance measures that are comparable among different estimation techniques is the development of a set of definitions that enforce a consistent interpretation of the vehicle trajectories. The subject of trajectory-based definitions is treated in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results, with additional material presented in Chapter 24, Concepts: Supplemental.

3. PEDESTRIAN MODE

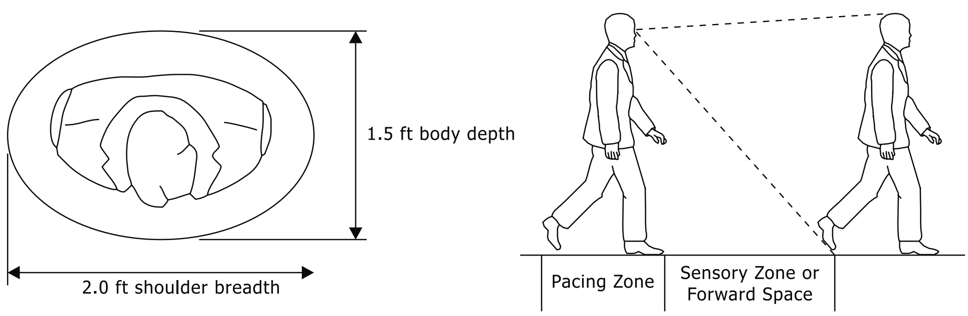
PEDESTRIAN CHARACTERISTICS

Pedestrian Space Requirements

Pedestrian facility designers use body depth and shoulder breadth for minimum space standards, at least implicitly. A simplified body ellipse of 1.5 ft by 2 ft, with a total area of 3 ft², is used as the basic space for a single pedestrian, as shown in Exhibit 4-11(a). This represents the practical minimum for standing pedestrians. In evaluating a pedestrian facility, an area of 8 ft² is used as the buffer zone for each pedestrian.

A walking pedestrian requires a certain amount of forward space. This forward space is a critical dimension, since it determines the speed of the trip and the number of pedestrians able to pass a point in a given time period. The forward space in Exhibit 4-11(b) is categorized into a pacing zone and a sensory zone (*8*).

Exhibit 4-11
Pedestrian Body Ellipse for Standing Areas and Pedestrian Walking Space Requirement



(a) Pedestrian Body Ellipse (b) Pedestrian Walking Space Requirement

Source: Adapted from Fruin (*8*).

Walking Speed

Factors affecting walking speed.

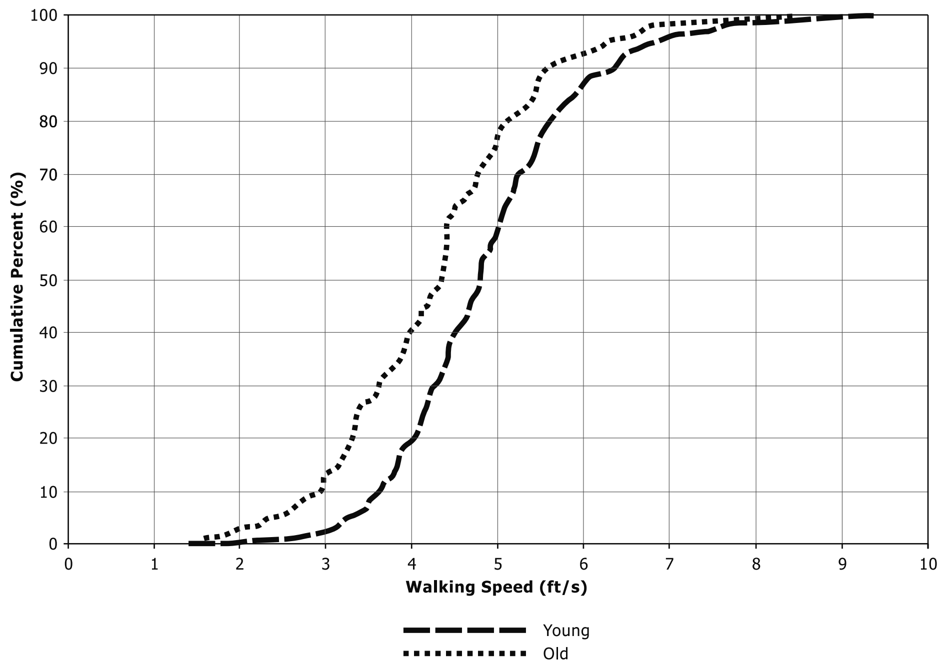
Pedestrian walking speed is highly dependent on the characteristics of the walking population. The proportion of elderly pedestrians (65 years old or more) and children in the population, as well as trip purpose, affect walking speed. In a national study (*9*), the average walking speed of younger (age 13–60) pedestrians crossing streets was found to be significantly different from that of older pedestrians (4.74 ft/s versus 4.25 ft/s, respectively). The 15th percentile speed, the speed used in the *Manual on Uniform Traffic Control Devices* (*10*) for timing the pedestrian clearance interval at traffic signals, was 3.03 ft/s for older pedestrians and 3.77 ft/s for younger pedestrians. Exhibit 4-12 shows these relationships.

For flow analysis, a default free-flow speed (i.e., an average pedestrian's speed on an otherwise empty sidewalk) of 5.0 ft/s (*11*) is appropriate for sidewalks and walkways, on the basis of average walking speeds. In calculating pedestrian crossing times, the 15th percentile crossing speed should be used. If no more than 20% of pedestrians are elderly, a crossing speed of 3.5 ft/s should be used (*11*). If elderly people constitute more than 20% of the total pedestrians, a crossing speed of 3.0 ft/s should be used. Several other factors may reduce

average pedestrian speed, such as grades over 5% or a high percentage of slow-walking children, and should be taken into consideration.

Exhibit 4-12
Observed Older and Younger Pedestrian Walking Speed Distribution at Unsignalized Intersections

LIVE GRAPH
Click here to view



Source: Adapted from *TCRP Report 112/NCHRP Report 562* (*9*).

Pedestrian Start-Up Time

At crosswalks located at signalized intersections, pedestrians may not step off the curb immediately when the WALK indication appears, in part because of perception–reaction time and in part to make sure that no vehicles have, or are about to, move into the crosswalk area. A pedestrian start-up time of 3 s is a reasonable midrange value to use for evaluating crosswalks at traffic signals.

PEDESTRIAN FLOW PARAMETERS

Speed, Flow, and Density Relationships

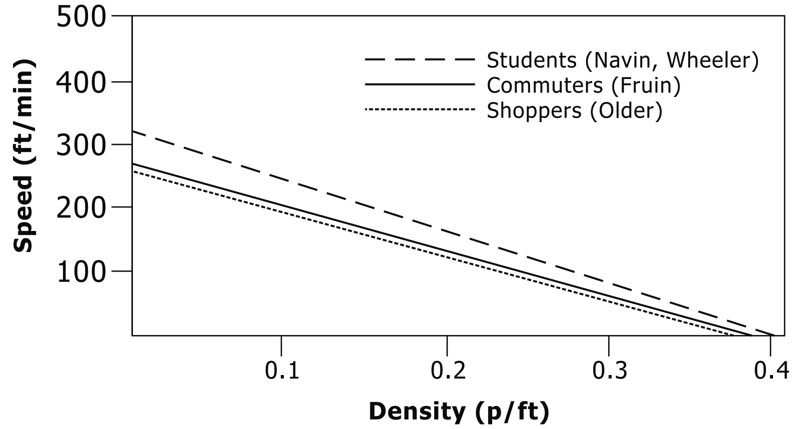
Speed–Density Relationships

The fundamental relationship between speed, density, and volume for directional pedestrian flow on facilities with no cross-flows, where pedestrians are constrained to a fixed walkway width (because of walls or other barriers), is analogous to that for vehicular flow. As volume and density increase, pedestrian speed declines. As density increases and pedestrian space decreases, the degree of mobility afforded to the individual pedestrian declines, as does the average speed of the pedestrian stream.

Exhibit 4-13 shows the relationship between speed and density for three pedestrian classes.

Exhibit 4-13
Relationships Between Pedestrian Speed and Density

LIVE GRAPH
Click here to view



Source: Adapted from Pushkarev and Zupan (*12*).

Flow–Density Relationships

Similarities of pedestrian movement to vehicular traffic.

The relationship among density, speed, and directional flow for pedestrians is similar to that for vehicular traffic streams and is expressed in Equation 4-11:

Equation 4-11

$$v_{ped} = S_{ped} \times D_{ped}$$

where

v_{ped} = unit flow rate (p/min/ft),

S_{ped} = pedestrian speed (ft/min), and

D_{ped} = pedestrian density (p/ft²).

The flow variable in Equation 4-11 is the unit width flow, defined earlier. An alternative, more useful, expression uses the reciprocal of density, or *space*:

Equation 4-12

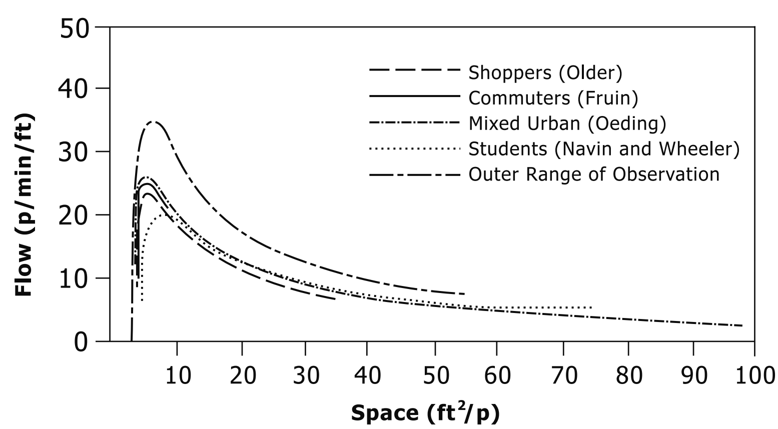
$$v_{ped} = \frac{S_{ped}}{M}$$

where M = pedestrian space (ft²/p).

The basic relationship between flow and space, recorded by several researchers, is illustrated in Exhibit 4-14:

Exhibit 4-14
Relationships Between Pedestrian Flow and Space

LIVE GRAPH
Click here to view



Source: Adapted from Pushkarev and Zupan (*12*).

The conditions at maximum flow represent the capacity of the walkway facility. From Exhibit 4-14, it is apparent that all observations of maximum unit flow fall within a narrow range of density, with the average space per pedestrian varying between 5 and 9 ft^2/p. Even the outer range of these observations indicates that maximum flow occurs at this density, although the actual flow in this study is considerably higher than in the others. As space is reduced to less than 5 ft^2/p, the flow rate declines precipitously. All movement effectively stops at the minimum space allocation of 2 to 4 ft^2/p.

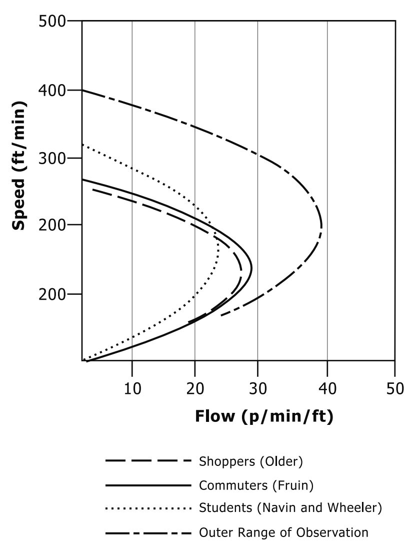
These relationships show that pedestrian traffic can be evaluated qualitatively by using basic concepts similar to those of vehicular traffic analysis. At flows near capacity, an average of 5 to 9 ft^2/p is required for each moving pedestrian. However, at this level of flow, the limited area available restricts pedestrian speed and freedom to maneuver.

Speed–Flow Relationships

Exhibit 4-15 illustrates the relationship between pedestrian speed and flow. These curves, similar to vehicle flow curves, show that when there are few pedestrians on a walkway (i.e., low flow levels), there is space available to choose higher walking speeds. As flow increases, speeds decline because of closer interactions among pedestrians. When a critical level of crowding occurs, movement becomes more difficult, and both flow and speed decline.

Exhibit 4-15
Relationships Between Pedestrian Speed and Flow

LIVE GRAPH
Click here to view

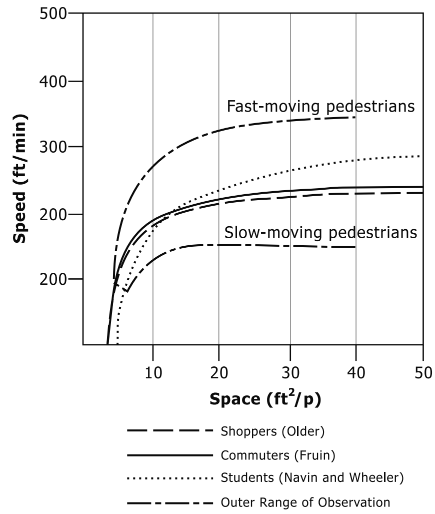


Source: Adapted from Pushkarev and Zupan (*12*).

Speed–Space Relationships

Exhibit 4-16 also confirms the relationships of walking speed and available space. The outer range of observations shown in Exhibit 4-16 indicates that at an average space of less than 15 ft^2/p, even the slowest pedestrians cannot achieve their desired walking speeds. Faster pedestrians, who walk at speeds of up to 350 ft/min, are not able to achieve that speed unless the average space is 40 ft^2/p or more.

Exhibit 4-16
Relationships Between Pedestrian Speed and Space



Source: Adapted from Pushkarev and Zupan (*12*).

Flow on Urban Sidewalks and Walkways

While the fundamental relationships described above hold for pedestrians on constrained facilities with linear flow (e.g., bridges and underground passageways), they are complicated on urban sidewalks and walkways by other factors. In particular, cross-flows, stationary pedestrians, and the potential for spillover outside of the walkway affect pedestrian flows on these facilities. Quantitative research describing the effects of these factors on pedestrian flow is limited, but the effects are described qualitatively here.

Cross-flows of pedestrians entering or exiting adjacent businesses, getting on or off buses at bus stops, or accessing street furniture are typical on most urban pedestrian facilities. Where pedestrian volumes are high, these cross-flows will disrupt the speed–flow relationships described above, resulting in lower pedestrian speeds at equivalent flow rates. In addition to cross-flows, stationary pedestrians will be present on most urban pedestrian facilities, as pedestrians stop to talk, look in store windows, or pick up a newspaper from a vending machine. Stationary pedestrians reduce pedestrian flow by requiring pedestrians to maneuver around them and decreasing the available width of the walkway.

Finally, in situations where pedestrians are not physically confined within the walkway, pedestrians will often choose to walk outside of the prescribed walking area (e.g., walk in the furniture zone or street) when high densities are reached. Thus, in practice, facilities will often break down, with pedestrians spilling over into the street, before the maximum flow rate shown in Exhibit 4-14 is reached. Therefore, typical practice is to design pedestrian facilities for LOS C or D densities (*13*).

The result of the combination of factors described above is that many pedestrian facilities will reach effective failure at densities far less than the facility's capacity. Analysis of pedestrian facilities should take into consideration local conditions, including the presence of destinations along the facility that contribute to cross-flows and stationary pedestrians, as well as opportunities for pedestrians to spill over onto adjacent facilities.

Pedestrian Type and Trip Purpose

The analysis of pedestrian flow is generally based on the mean, or average, walking speeds of groups of pedestrians. Within any group, or among groups, there can be considerable differences in flow characteristics due to trip purpose, adjacent land use, type of group, age, mobility, cognitive ability, and other factors.

Pedestrians going to and from work and using the same facilities day after day walk at higher speeds than do shoppers, as was shown in Exhibit 4-13. Older or very young persons tend to walk more slowly than do other groups. Shoppers not only tend to walk more slowly than do commuters but also can decrease the effective walkway width by stopping to window-shop and by carrying shopping bags. The analyst should adjust for pedestrian behavior that deviates from the regular patterns represented in the basic speed, volume, and density curves.

Influences of Pedestrians on Each Other

Photographic studies show that pedestrian movement on sidewalks is affected by other pedestrians, even when space is more than 40 ft^2/p. At 60 ft^2/p, pedestrians have been observed walking in a checkerboard pattern, rather than directly behind or alongside each other. The same observations suggest the necessity of up to 100 ft^2/p before completely free movement occurs without conflicts, and that at 130 ft^2/p, individual pedestrians are no longer influenced by others (*14*). Bunching or platooning does not disappear until space is about 500 ft^2/p or higher.

Maintaining flow in the minor (opposing) direction.

Another issue is the ability to maintain flow in the minor direction on a sidewalk when opposed by a major pedestrian flow. For pedestrian streams of roughly equal flow in each direction, there is little reduction in the capacity of the walkway compared with one-way flow, because the directional streams tend to separate and occupy a proportional share of the walkway. However, if the directional split is 90% versus 10% and space is 10 ft^2/p, capacity reductions of about 15% have been observed. This reduction results from the minor flow using more than its proportionate share of the walkway.

Opposing flows on stairways.

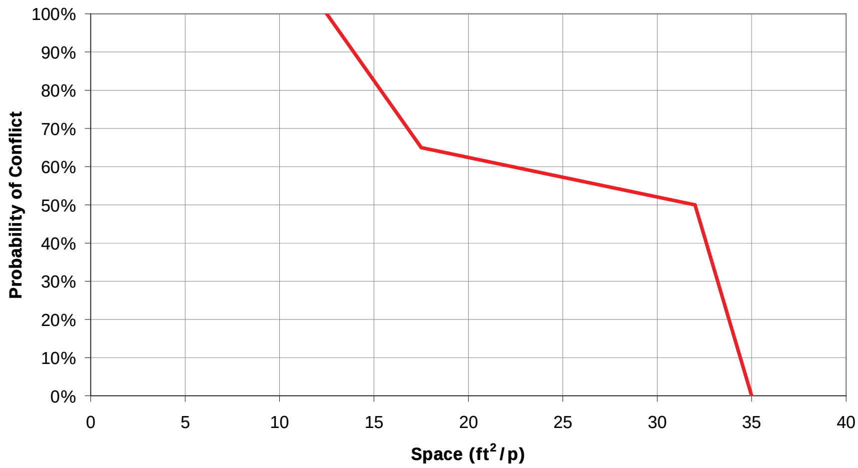
Similar, but more severe, effects are seen with stairways. In contrast to their behavior on a level surface, people tend to walk in lines or lanes in traversing stairs. A small reverse flow occupies one pedestrian lane (30 in.) of the stair's width. For a stair 60 in. (5 ft) wide, a small reverse flow could consume half its capacity (*13*).

Cross-flows.

A pedestrian's ability to cross a pedestrian stream is impaired at space values less than 35 to 40 ft^2/p, as shown in Exhibit 4-17 (*8*). Above that level, the probability of stopping or breaking the normal walking gait is reduced to zero. Below 15 ft^2/p, virtually every crossing movement encounters a conflict. Similarly, the ability to pass slower pedestrians is unimpaired above 35 ft^2/p, but it becomes progressively more difficult as space allocations drop to 18 ft^2/p, the point at which passing becomes virtually impossible.

Exhibit 4-17
Probability of Conflict Within Pedestrian Cross-Flows

LIVE GRAPH
Click here to view



Source: Adapted from Fruin (*8*).

Pedestrian Facility Characteristics

Effective Walkway Width

The lane concept used for highway analysis is frequently not applicable to pedestrian analysis, because studies have shown that pedestrians normally do not walk in organized lanes. The concept is meaningful, however, in the following situations:

- Determining how many pedestrians can walk abreast in a given walkway width—for example, in establishing the minimum sidewalk width that will permit two pedestrians to pass each other conveniently; and
- Determining the capacity of a stairway, since pedestrians will tend to organize into lanes on stairways.

In other situations, the capacity of a pedestrian facility is directly related to the width of the facility. However, not all of the facility's width may be usable, because of obstructions and pedestrians' tendencies to shy away from curbs and building walls. The portion of a pedestrian facility's width that is used for pedestrian circulation is called the *effective width*. The degree to which single obstructions, such as poles, signs, and hydrants, influence pedestrian movement and reduce effective walkway width is not extensively documented. Although a single point of obstruction would not reduce the effective width of an entire walkway, it would affect its immediate vicinity.

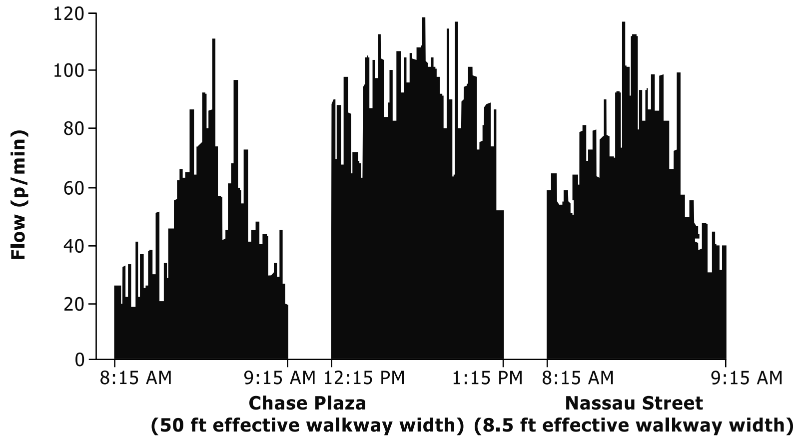
To avoid interference when two pedestrians pass each other, each should have at least 2.5 ft of walkway width (*12*). When pedestrians who know each other walk close together, each occupies an average width of 26 in., allowing considerable likelihood of contact due to body sway. Lateral spacing less than this occurs only in the most crowded situations.

Pedestrian Platoons

Average pedestrian flow rates are of limited usefulness unless reasonable time intervals are specified. Exhibit 4-18 illustrates that average flow rates can be misleading. The data shown are for two locations in New York City, but the pattern is generally characteristic of concentrated central business districts. Flows

during a 1-min interval can be more than double the rate in another, particularly at relatively low flows. Even during the peak 15-min periods, the peak 1-min flow exceeded the average flow by at least 20% and sometimes up to 75%.

Exhibit 4-18
Minute-by-Minute Variations in Pedestrian Flow

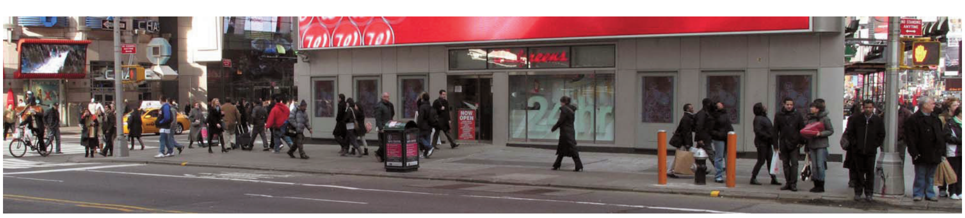


Source: Adapted from Pushkarev and Zupan (*12*).

Depending on traffic patterns, a facility designed for average flow can afford a lower quality of flow for a portion of its pedestrian traffic. However, it is not prudent to design for extreme peak 1-min flows that occur only 1% or 2% of the time. A relevant time period should be determined through closer evaluation of the short-term fluctuations of pedestrian flow.

Short-term fluctuations are present in most unregulated pedestrian traffic flows because of the random arrivals of pedestrians. On sidewalks, these random fluctuations are exaggerated by the interruption of flow and queue formation caused by traffic signals. Transit facilities can create added surges in demand by releasing large groups of pedestrians in short time intervals, followed by intervals during which no flow occurs. Until they disperse, pedestrians in these types of groups move together as a platoon (Exhibit 4-19). Platoons can also form when passing is impeded because of insufficient space and faster pedestrians must slow down behind slower-moving ones.

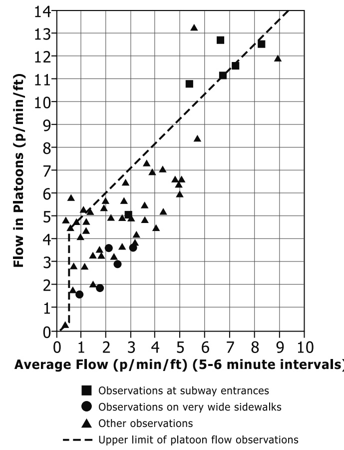
Exhibit 4-19
Platoon Flow on a Sidewalk



The scatter diagram shown in Exhibit 4-20 compares the platoon flow rate (i.e., the rate of flow within platoons of pedestrians) with the average flow rate for durations of 5 to 6 min. The dashed line approximates the upper limit of platoon flow observations.

Exhibit 4-20
Relationship Between Platoon Flow and Average Flow

LIVE GRAPH
Click here to view



Source: Adapted from Pushkarev and Zupan (*12*).

CAPACITY CONCEPTS

Pedestrian Circulation Facilities

Pedestrian capacity on facilities designed for pedestrian circulation is typically expressed in terms of *space* (square feet per pedestrian) or *unit flow* (pedestrians per minute per foot of walkway width). The relationship between space and flow was illustrated in Exhibit 4-14. Capacity occurs when the maximum flow rate is achieved. Typical values for pedestrian circulation facilities are as follows:

- Walkways with random flow, 23 p/min/ft;
- Walkways with platoon flow (average over 5 min), 18 p/min/ft;
- Cross-flow areas, 23 p/min/ft (sum of both flows); and
- Stairways (up direction), 15 p/min/ft.

As shown in Exhibit 4-15, average pedestrian speeds at capacity are about half the average speed obtained under less congested conditions. As a result, pedestrian circulation facilities are typically not designed for capacity but rather for a less congested condition that achieves lower pedestrian throughput but that provides pedestrians with greater opportunity to travel at their desired speed with minimal conflicts with other pedestrians. Moreover, as described above under "Flow on Urban Sidewalks and Walkways," pedestrian facilities often break down before maximum flow rates are achieved, as a result of pedestrian spillover outside of the walkway into the furniture zone or roadway.

Pedestrian Queuing Facilities

Pedestrian capacity on facilities designed for pedestrian queuing is expressed in terms of space (square feet per pedestrian). In a queuing area, the pedestrian stands temporarily while waiting to be served. In dense, standing crowds, there is little room to move, but limited circulation is possible as the average space per pedestrian increases. Queuing at or near capacity (2 to 3 ft^2/p) typically occurs only in the most crowded elevators or transit vehicles. Queuing on sidewalks, waiting to cross at street corners, is more typically in the 3 to 6 ft^2/p range, which is still crowded but provides some internal maneuverability.

4. BICYCLE MODE

BICYCLE FLOW PARAMETERS

Although bicyclists are not as regimented as vehicles, they tend to operate in distinct lanes of varying widths when space is available. The capacity of a bicycle facility depends on the number of effective lanes used by bicycles. Shared-lane facilities typically have only one effective lane, but segregated facilities such as bicycle lanes, shoulder bikeways, pathways, and cycle tracks may have more than one effective lane, depending on their width. When possible, an analysis of a facility should include a field evaluation of the number of effective lanes in use. When this is not possible, or when future facilities are planned, a standard width for an effective bicycle lane is approximately 4 ft (*15*). The American Association of State Highway and Transportation Officials recommends that off-street bicycle paths be 10 ft wide (*15*).

Research demonstrates that three-lane bicycle facilities operate more efficiently than two-lane bicycle facilities, affording considerably better quality of service to users (*16*). The improved efficiency is due primarily to increased opportunities for passing and for maneuvering around other bicyclists and pedestrians. This reinforces the value of determining the number of effective lanes as the principal input for analyzing a bicycle facility.

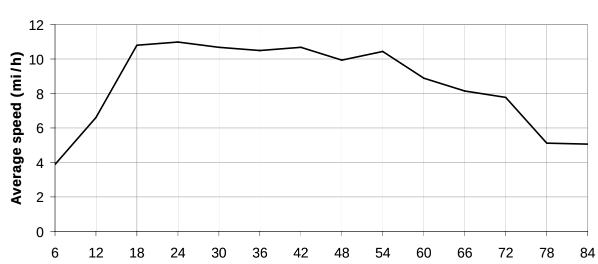
A study that compared mean bicycle speeds with bicycle flow rates over 5-min periods found at most a minor effect of flow rates on speed, for flow rates ranging from 50 to 1,500 bicycles/h. When the analysis focused on platoons of bicycles with headways less than 5 s, bicycle speeds trended slightly lower as flow rates increased (*17*).

Most bicyclists travel on facilities that are shared with automobiles. In these circumstances, bicycle flow is significantly affected by the characteristics of surrounding automobile flow. Bicyclists often must wait behind queues of automobiles. Even where bicyclists may pass such queues, they are often forced to slow because the available space in which to pass is too constrained to allow free-flow speeds to occur.

Data collected for more than 400 adult bicyclists riding on uninterrupted multiuse segments showed an average speed of 12.8 mi/h (*16*). However, the speed of an individual bicyclist varies considerably from this average on the basis of trail conditions, age, fitness level, and other factors. Exhibit 4-21 shows how bicyclist speed varies with age, on the basis of Danish data. Data are for typical bicyclists on flat terrain.

Exhibit 4-21
Age Effects on Bicyclist Speed

LIVE GRAPH
Click here to view



Source: Danish Road Directorate (*18*).

Flow rates of bicyclists usually vary over the course of an hour. As described above for automobiles, HCM analyses typically consider the peak 15 min of flow during the analysis hour. Because inputs to HCM procedures are typically expressed in terms of hourly demands, the HCM uses the PHF, shown by Equation 4-1, to convert an hourly volume into a peak 15-min flow rate. Data for bicycles on eight trails, recorded over three separate time periods for each trail, showed PHFs ranging from 0.70 to 0.99, with an average of 0.85 (*16*).

CAPACITY CONCEPTS

Because service quality deteriorates at flow levels well below capacity, the concept of capacity has little utility in the design and analysis of bicycle paths and other facilities. Capacity is rarely observed on bicycle facilities. Values for capacity, therefore, reflect sparse data, generally from Europe and generally extrapolated from flow rates over time periods substantially less than 1 h.

One study reported capacity values of 1,600 bicycles/h/ln for two-way bicycle facilities and 3,200 bicycles/h/ln for one-way facilities. Both values were for exclusive bicycle facilities operating under uninterrupted-flow conditions (*19*). Other studies have reported values in the range of 1,500 to 5,000 bicycles/h/ln for one-way uninterrupted-flow facilities (*16*).

Danish guidelines suggest that bicycle capacity is normally only relevant at signalized intersections in cities and that a rule of thumb for the capacity of a two-lane cycle track is 2,000 bicycles/h under interrupted-flow conditions (i.e., 1,000 bicycles/h/ln) (*20*). The HCM recommends a saturation flow rate of 2,000 bicycles/h/ln for a one-direction bicycle lane under interrupted-flow conditions, which is equivalent to a capacity of 1,000 bicycles/h/ln when the bicycle lane receives a green indication during 50% of the signal cycle.

DELAY

Delay is an important performance measure for bicyclists on interrupted-flow system elements. This is true because delay increases travel time and because the physical exertion required to accelerate a bicycle makes stopping or slowing undesirable and tiring. The difficulty involved in stopping and starting a bicycle often makes it appropriate to assess not only the control delay incurred by bicyclists but also the number of stops that bicyclists are required to make to traverse a facility. For instance, a facility with STOP signs every several hundred feet will require bicyclists to stop frequently and thus will provide lower capacity and quality of service to users.

5. TRANSIT MODE

BUS SPEED PARAMETERS

Material in this section generally refers to buses but is also applicable to streetcars and light rail vehicles operating on urban streets, except where specifically stated otherwise.

Bus speeds on urban streets are influenced by the same factors that influence automobile speeds, particularly the delay caused by traffic signals and other forms of intersection control. As heavy vehicles, buses accelerate and decelerate more slowly than passenger cars. In addition, there are many bus-specific factors that influence speed, involving operations, vehicle, roadway, and passenger characteristics. These factors are described below.

Operations Characteristics

Stop Spacing

Unlike other urban street users, most transit vehicles (except for express buses) stop periodically so that passengers may board and alight. Each stop introduces up to four forms of delay:

- *Acceleration and deceleration delay,* as a bus slows down approaching a stop and returns to its running speed after departing the stop;
- *Passenger service time,* while the bus is stopped and passengers are actively boarding and alighting;
- *Traffic signal delay,* when a bus arrives at a near-side stop (i.e., a bus stop immediately prior to an intersection) during the green interval, serves its passengers, and then experiences delay while waiting for the traffic signal to turn green again before proceeding; and
- *Reentry time,* when a bus stops out of the curb travel lane and then must wait for a gap in traffic before it can reenter the lane.

Increasing the stop spacing reduces the number of occurrences of these types of delay, which results in a net increase in speeds. (Passenger service times may increase, though, as passenger activity is concentrated at fewer stops.) For example, five bus rapid transit routes in Los Angeles that have stop spacings ranging from 2/3 to 1 mi and no other contributing factors (e.g., transit signal priority) achieved travel time reductions of 17% to 29% compared with the previous local bus service (*21*).

The ability to increase stop spacing depends on many factors, including the surrounding pedestrian environment, the locations of transit trip generators and transfer opportunities, and driveway and curb parking locations (*13*).

Fare Payment

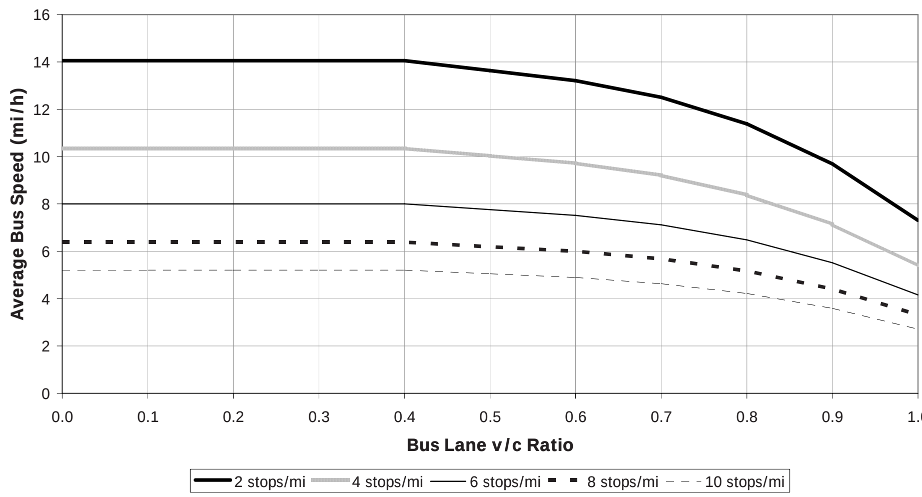
The time required for passengers to pay a fare affects the passenger service time at stops. The average base amount of time needed to board a high-floor bus with no or prepaid (e.g., bus pass or free transfer) fare payment is 2.5 s/passenger. The various types of fare payment methods (e.g., cash, tickets, tokens, magnetic-stripe cards, smart cards) have service times associated with them that increase the service time by up to 1.8 s/passenger, on average, above the base level (*13*).

Service Planning and Scheduling

Bus speeds along an urban street decline when 50% or more of a bus lane's hourly capacity is utilized, as illustrated in Exhibit 4-22. As the number of buses using a bus lane increases, there is a greater probability that one bus will delay another bus, either by using all available space at a bus stop or by requiring bus passing and weaving maneuvers. At a volume-to-capacity (*v/c*) ratio of 1.0, bus speeds are approximately half of what can be achieved at *v/c* ratios under 0.5 (*13*).

Exhibit 4-22
Illustrative Bus Speed Relationship to Bus Lane *v/c* Ratio

LIVE GRAPH
Click here to view



Notes: Assumes 30-s dwell times, downtown bus lane with right-turn lanes, and typical signal timing.
Source: *TCRP Report 100: Transit Capacity and Quality of Service Manual* (*13*).

On buses where demand exceeds seating capacity, causing some passengers to stand, more passenger service time is required at stops, because standing passengers must push toward the back of the bus to allow other passengers to board and because alighting passengers take longer to get to a door.

Vehicle Characteristics

Low-floor buses decrease passenger service time by eliminating the need to ascend and descend steps. These buses are particularly useful for routes frequently used by the elderly, persons with disabilities, or persons with strollers or bulky carry-on items. Wide bus doors also allow more passengers to board and alight simultaneously (*13*).

Roadway Characteristics

Transit Preferential Treatments

Refer to TCRP Report 118 *(22) for illustrations and guidance on appropriate locations for transit preferential treatments.*

Transit preferential treatments, both geometric and operational, give transit vehicles a travel time advantage over other roadway users. Common types of treatments include the following:

- *Exclusive bus lanes.* One or more lanes on a street that are reserved on a full- or part-time basis for the exclusive use of buses. Exclusive bus lanes partially or entirely eliminate interactions with automobiles and other roadway users that can slow down a bus. With typical signal timing, bus

lanes typically provide a 1.0- to 1.8-min/mi speed advantage over mixed-traffic operations (*13*).

- *Queue jumps.* A short bus lane section (often shared with a right-turn lane), in combination with an advance green indication for the lane, that allows buses to move past a queue of cars at a signal. These typically reduce bus delay at a signalized intersection by 5% to 15% (*22*).
- *Curb extensions.* An extension of the sidewalk to the edge of the travel or bicycle lane (e.g., by removing on-street parking). Curb extensions allow buses to stop in the travel lane and thus to avoid delay when they leave the stop. At the range of curb volumes appropriate for curb extensions (under 500 veh/h), they can save buses up to 5 s of delay per stop on average (*22*).
- *Traffic signal priority*. Changes to traffic signal timing to favor bus operations. These include *passive* (pretimed) changes—for example, to set a street's signal progression to favor buses rather than cars (as is done in downtown Ottawa). *Active* strategies adjust signal timing in reaction to the arrival of a bus—for example, to extend a green interval to accommodate a bus arriving late in the interval or to shorten the red interval to get a stopped bus under way sooner. *Real-time* strategies consider both automobile and bus arrivals at a single intersection or a network of intersections. *Traffic signal preemption* is rarely used for buses because of signal system operations and safety issues but is a feature of some light rail routes. Systems of intersections equipped with traffic signal priority have typically reduced transit travel times by 5% to 15%, with a range of reported values of 0% to 49% (*13*, *22*).
- *Turn restriction exemptions*. Buses are allowed to make turns at locations where other vehicles are not allowed to. This treatment allows buses to travel more direct routes; the time saved depends on the length of and the delay associated with the alternate route (*13*).

Bus Stop Location

Buses can stop in the travel lane (*on-line*) or out of the travel lane (*off-line*), for example, in a bus pullout. Off-line stops require buses to find a gap in traffic when they reenter the travel lane, which generates reentry delay that can average 1 to 15 s with random vehicle arrivals, depending on the traffic volume in the curb lane. Some jurisdictions have passed yield-to-bus laws that require motorists to allow a bus back into traffic. These laws can reduce or eliminate reentry delay, depending on the degree of motorist compliance (*13*).

Bus stops can be located before an intersection (*near-side*), after an intersection (*far-side*), or between intersections (*midblock*). The interaction of right-turning vehicles and buses at near-side stops can create delays. Buses are also more likely to get caught by a red signal after serving passengers at a near-side stop than at a far-side stop. On the other hand, buses using on-line far-side stops may cause traffic to back up behind the bus into the intersection. Refer to the *Transit Capacity and Quality of Service Manual* (*13*) for a full list of the advantages and disadvantages of each type of stop location.

Passenger Characteristics

Passenger Distribution

The distribution of boarding passengers among bus stops affects the passenger service time of each stop. If passenger boardings are concentrated at one stop along a street, that street's bus capacity will be lower than if boardings were more evenly distributed. Bus stop capacity decreases as average passenger service time increases, and the bus capacity of a street is constrained by the lowest bus stop capacity along the street. With a lower capacity, it takes fewer scheduled buses in an hour for interactions between buses to begin to affect bus speeds. However, if boardings are concentrated because of a consolidation of bus stops, the net effect will be improved speed and a reduced number of stops, which will offset the increased passenger service time at the remaining stops.

Strollers, Wheelchairs, and Bicycles

Passenger service times are longer for passengers with strollers or using wheelchairs, particularly with high-floor buses when a lift must be deployed. A passenger using a bicycle rack mounted to the bus will also cause service time to increase, except when other passengers are still being served after the bicycle has been secured. In many cases, these events are sufficiently infrequent to be indistinguishable from the normal variation in passenger demands and service times at a bus stop.

CAPACITY CONCEPTS

Differences Between Transit and Automobile Capacity

Transit capacity is different from highway capacity: it deals with the movement of both people and vehicles, it depends on the size of the transit vehicles and how often they operate, and it reflects the interaction of passenger traffic and vehicle flow. Transit capacity depends on the operating policy of the transit agency, which specifies service frequencies and allowable passenger loadings. Accordingly, the traditional concepts applied to highway capacity must be adapted and broadened.

Two key characteristics differentiate transit from the automobile in terms of availability and capacity. Although the automobile has widespread access to roadway facilities, transit service is available only in certain locations and during certain times. Roadway capacity is available 24 h/day once constructed, but transit passenger capacity is limited by the number of transit vehicles operated at a given time.

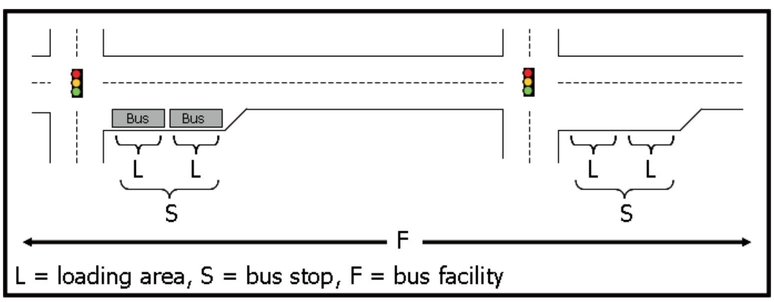
Throughout the transit sections of the HCM, a distinction is made between vehicle and person capacity. *Vehicle capacity* reflects the number of transit units (buses or trains) that pass a given location during a given time period. Thus, bus vehicle capacity is most closely analogous to automobile capacity. *Person capacity* reflects the number of people that can be carried past a given location during a given time period under specified operating conditions, without unreasonable delay, hazard, or restriction, and with reasonable certainty.

On-street bus capacity has a two-dimensional nature. It is possible to operate many buses, each carrying only a few passengers. Whether the buses are full or empty, a larger number of buses can have a negative impact on LOS in terms of highway capacity. Alternatively, only a few buses could operate, each overcrowded. This represents a poor quality of service from the passenger perspective, and long waiting times would detract from user convenience.

Vehicle Capacity

Vehicle (bus) capacity is commonly determined for three locations along an urban street: individual loading areas (berths) at bus stops, individual bus stops, and an urban street facility, as illustrated in Exhibit 4-23. Each location directly influences the next. The vehicle capacity of a bus stop is controlled by the vehicle capacities of the loading areas, and the vehicle capacity of the urban street facility is controlled by the vehicle capacity of the critical stop within the facility.

Exhibit 4-23
Bus Loading Areas, Stops, and Facilities



Source: *TCRP Report 100: Transit Capacity and Quality of Service Manual* (*13*).

Loading Area Capacity

The main elements that determine loading area capacity are dwell time, dwell time variability, traffic signal timing, failure rate, and clearance time (*13*):

- *Dwell time* is the sum of passenger service time and the time to open and close the bus doors.
- *Dwell time variability* accounts for the fact that buses do not stop for the same amount of time at a stop because of fluctuations in passenger demand for buses and routes.
- *Traffic signal timing* affects the time available in an hour for buses to enter and exit a bus stop. In addition, the length of red in relation to the dwell time of a bus affects vehicle capacity: if passenger movements have finished but the vehicle must wait for the traffic signal to turn green, vehicle capacity will be less than if the bus can leave immediately so that another bus can use the loading area.
- *Failure rate* is a design input and is the probability that one bus will arrive at a bus stop only to find all loading areas already occupied.
- *Clearance time* is an interval after a bus is ready to depart during which the loading area is not available for use by a following bus. Part of this time is fixed, consisting of the time for a bus to start up and travel its own length, clearing the stop. For off-line stops, a second component of clearance time is the reentry time discussed previously.

Bus Stop Capacity

Bus stops consist of one or more loading areas. When a bus stop consists of a single loading area, its capacity is equivalent to the loading area capacity. However, when a bus stop consists of multiple loading areas, the number of loading areas and the design of the loading areas also influence its capacity.

Effective loading areas.

The vast majority of on-street bus stops are *linear* bus stops, where the first bus to arrive occupies the first loading area, the second bus occupies the second loading area, and so on. Each additional linear loading area at a bus stop is less efficient than the one before it for three reasons:

1. The rear loading areas will be used less often than the first loading area;
2. Not knowing which loading area their bus will stop at, passengers may have to walk down the line of buses to get to their bus, increasing its passenger service time; and
3. Depending on how closely buses stop behind the bus in front and the buses' ability to pass one another, a bus may not be able to leave its loading area until the bus in front of it departs.

Efficiency drops significantly above three loading areas. Efficiency is also affected by whether the bus stop is on-line or off-line and by whether buses arrive irregularly (the usual situation) or are operated in groups of two or three buses (*platoons*) that act like cars of a train (*13*).

Bus Facility Capacity

Bus facility capacity is constrained by the capacity of the critical bus stop along the facility. The critical bus stop is usually the bus stop with the longest dwell time; however, high vehicular traffic volumes (particularly turning volumes) along with conflicting pedestrian volumes can combine to make a particular bus stop the critical stop. The impact of vehicular traffic on capacity also depends on the location of a particular bus stop, with near-side stops being affected more than far-side stops, and on whether buses operate in mixed traffic or in a bus lane (*13*).

Exhibit 4-24 summarizes the elements that determine bus facility capacity.

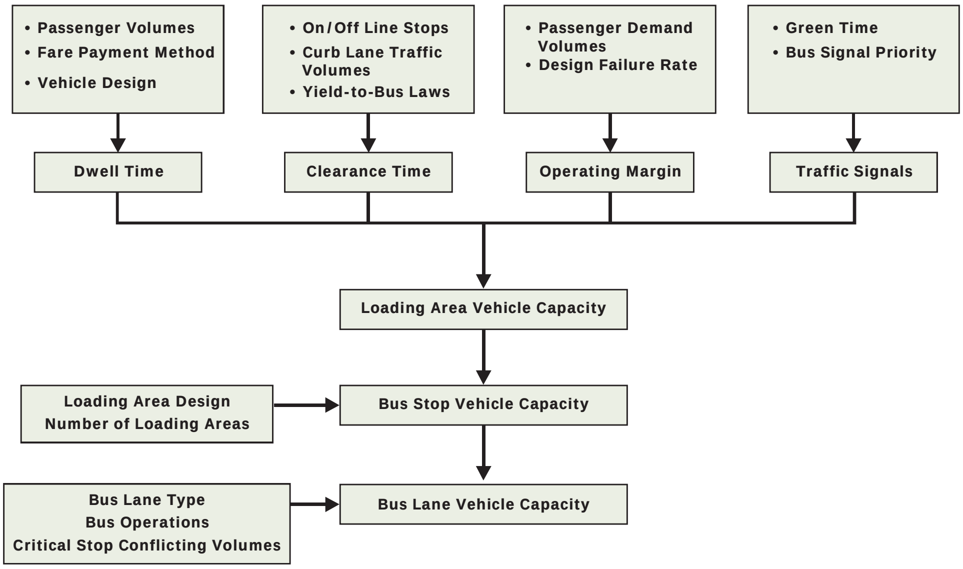
Person Capacity

For HCM analysis purposes, person capacity is typically calculated only at the facility level.

Person capacity is determined by three main factors (*13*):

1. *Vehicle capacity,* which determines the maximum number of buses that can be scheduled to use the bus facility over the course of an hour;
2. *Agency policy,* which sets loading standards for buses and determines how frequently buses operate (which is usually less than the maximum possible frequency); and
3. *Passenger demand characteristics,* reflected by a PHF.

Exhibit 4-24
Capacity Factors for Bus Facilities



Source: *TCRP Report 100: Transit Capacity and Quality of Service Manual* (*13*).

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CHAPTER 5
QUALITY AND LEVEL-OF-SERVICE CONCEPTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

There are many ways to measure the performance of a transportation facility or service—and many points of view that can be considered in deciding which measurements to make. The agency operating a roadway, automobile drivers, pedestrians, bicyclists, bus passengers, decision makers, and the community at large all have their own perspectives on how a roadway or service should perform and what constitutes "good" performance. As a result, there is no one right way to measure and interpret performance. **Chapter 5, Quality and Level-of-Service Concepts,** presents the concepts that the *Highway Capacity Manual* (HCM) uses to describe performance from the traveler point of view in a way that is designed to be useful to roadway operators, decision makers, and members of the community.

Q*uality of service* describes how well a transportation facility or service operates from the traveler's perspective. *Level of service* (LOS) is a quantitative stratification of a performance measure or measures that represent quality of service. The LOS concept facilitates the presentation of results, through the use of a familiar A (best) to F (worst) scale. LOS is defined by one or more *service measures* that both reflect the traveler perspective and are useful to operating agencies.

Three overarching concepts—quality of service, LOS, and service measures—are the subjects of this chapter.

2. QUALITY OF SERVICE

Quality of service defined.

Quality of service describes how well a transportation facility or service operates from a traveler's perspective. Quality of service can be assessed in a number of ways. Among them are directly observing factors perceivable by and important to travelers (e.g., speed or delay), surveying travelers, tracking complaints and compliments about roadway conditions, forecasting traveler satisfaction by using models derived from past traveler surveys, and observing services not directly perceived by travelers (e.g., average time to clear an incident) that affect measures they can perceive (e.g., speed or arrival time at work).

Factors that influence traveler perceived quality of service have been found to include

- Travel time, speed, and delay;
- Number of stops incurred;
- Travel time reliability;
- Maneuverability (e.g., ease of lane changing, percent time-spent-following other vehicles);
- Comfort (e.g., bicycle and pedestrian interaction with and separation from traffic, transit vehicle crowding, ride comfort);
- Convenience (e.g., directness of route, frequency of transit service);
- Safety (actual or perceived);
- User cost;
- Availability of facilities and services;
- Facility aesthetics; and
- Information availability (e.g., highway wayfinding signage, transit route and schedule information).

The HCM provides tools to measure the multimodal operations aspects of quality of service.

The HCM's scope, measuring the multimodal performance of highway and street facilities, is narrower than the list of quality-of-service aspects listed above. As discussed in Chapter 1, HCM User's Guide, companion documents to the HCM address highway safety, roadway design, and wayfinding signage, among other topics. The HCM focuses particularly on the travel time, speed, delay, maneuverability, and comfort aspects of quality of service, although a limited number of the HCM's performance measures address some of the other aspects listed above. Federal Strategic Highway Research Program 2 research that was programmed at the time this manual was written may add measures of travel-time reliability to the HCM in the future.

LOS is an important tool used by the HCM to stratify quality of service.

The HCM provides a variety of performance measures in Volumes 2 and 3 to assess the quality of service of transportation system elements. These measures can be directly observed in the field or estimated from things observable in the field. LOS is the stratification of quality of service and is further described in the next part of this chapter.

3. LEVEL OF SERVICE

DEFINITION

LOS defined.

LOS is a quantitative stratification of a performance measure or measures that represent quality of service. The measures used to determine LOS for transportation system elements are called *service measures.* The HCM defines six levels of service, ranging from A to F, for each service measure, or for the output from a mathematical model based on multiple performance measures. LOS A represents the best operating conditions from the traveler's perspective and LOS F the worst. For cost, environmental impact, and other reasons, roadways are not typically designed to provide LOS A conditions during peak periods, but rather some lower LOS that reflects a balance between individual travelers' desires and society's desires and financial resources. Nevertheless, during low-volume periods of the day, a system element may operate at LOS A.

LOS is measured on an A–F scale. LOS A represents the best conditions from a traveler's perspective.

USAGE

LOS is used to translate complex numerical performance results into a simple A–F system representative of travelers' perceptions of the quality of service provided by a facility or service. The LOS letter result hides much of the complexity of facility performance. This feature is intended to simplify decision making on whether facility performance is generally acceptable and whether a future change in performance is likely to be perceived as significant by the general public. The language of LOS provides a common set of definitions that transportation engineers and planners can use to describe operating conditions; however, it is up to local policy makers to decide the appropriate LOS for a given system element in their community. One reason for the widespread adoption of the LOS concept by agencies is the concept's ability to communicate roadway performance to nontechnical decision makers. However, LOS has other strengths and weaknesses, described below, that both analysts and decision makers need to be mindful of.

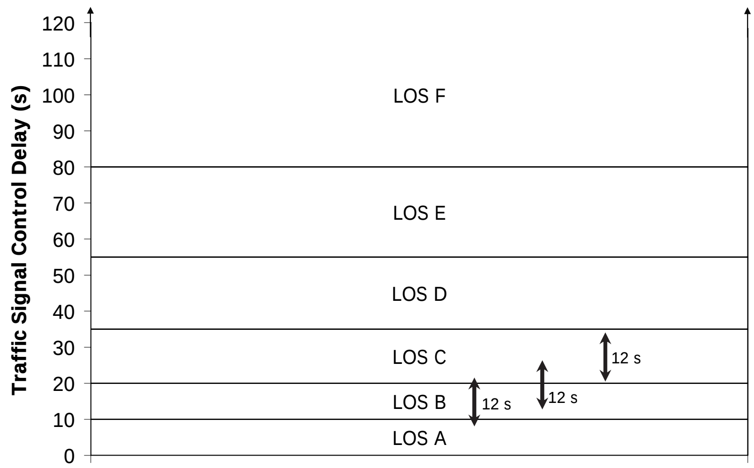
LOS is a useful and widely adopted tool for communicating roadway performance to laypersons and decision makers. However, one should also be mindful of its weaknesses.

Step Function Nature of LOS

LOS is a step function. An increase in average control delay of 12 s at a traffic signal, for example, may result in no change in LOS, a drop of one level, or even a drop of two levels, depending on the starting value of delay, as illustrated in Exhibit 5-1.

From a traveler perception standpoint, the condition shown in Exhibit 5-1 is not necessarily inconsistent. A change of LOS indicates that roadway performance has transitioned from one given range of traveler-perceivable conditions to another range, while no change in LOS indicates that conditions have remained within the same performance range as before. Service measure values indicate where conditions lie within a particular performance range. However, because a small change in a service measure, or the output from a mathematical model based on multiple performance measures, can sometimes result in a change from one LOS to another, the LOS result could imply a more significant effect than actually occurred.

Exhibit 5-1
Example of the Step Function Nature of LOS



Identical changes in the service measure value may result in no change in LOS or a change of one or more levels of service, depending on how close the starting value is to a LOS threshold.

Defining performance standards on the basis of LOS (or any fixed numerical value) means that small changes in performance can sometimes result in the standard being exceeded, when a facility is already operating close to the standard.

This aspect of LOS can be a particularly sensitive issue when agencies define their operational performance standards solely by using LOS. However, the definition of a fixed standard, whether numerically or as a LOS letter, always brings the possibility that a small change in performance can trigger the need for potentially costly improvements.

Variability of the Inputs to LOS

Although computer software that implements HCM methodologies can sometimes report results to many decimal places, three major sources of uncertainty influence service measure values and, thus, the LOS result:

Chapter 7, Interpreting HCM and Alternative Tool Results, discusses sources of uncertainty and their impacts on analysis results in more detail.

1. The models used to estimate service measure values have confidence intervals associated with their outputs;
2. These models may, in turn, rely on the output of other models that have their own associated confidence intervals; and
3. The accuracy of input variables, such as demand flow rate, is taken to be absolute when, in fact, there is a substantial stochastic variation around the measured values.

Models provide a best estimate of service measure values, but the "true" value likely lies within a confidence interval range above or below the estimated value.

Thus, any reported service measure value, whether resulting from an HCM methodology, an alternative tool, or even field measurement, potentially has a fairly wide range associated with it in which the "true" value actually lies. The LOS concept helps to downplay the implied accuracy of a numeric result by presenting a range of service measure results as being reasonably equivalent from a traveler's point of view. Nevertheless, the same variability issues also mean that the "true" LOS value may be different from the one predicted by a methodology. In addition, for any given set of conditions, different travelers may perceive their LOS to be different from one another, as well as different from the LOS estimated by an HCM method. One way of thinking about reported service measure values and the corresponding LOS result is that they are the statistical "best estimators" of conditions and aggregate traveler perception.

Beyond LOS F

The HCM uses LOS F to define operations that have either broken down (i.e., demand exceeds capacity) or have exceeded a specified service measure value (or combination of service measure values) that most users would consider unsatisfactory. However, particularly for planning applications where different alternatives may be compared, analysts may be interested in knowing just how bad the LOS F condition is. Several measures are available to describe individually, or in combination, the severity of a LOS F condition:

The HCM does not subdivide LOS F, but several measures are available to describe the severity of a LOS F condition.

- *Demand-to-capacity ratios* describe the extent to which capacity is exceeded during the analysis period (e.g., by 1%, 15%, etc.).
- *Duration of LOS F* describes how long the condition persists (e.g., 15 min, 1 h, 3 h).
- *Spatial extent measures* describe the areas affected by LOS F conditions. These include measures such as the back of queue and the identification of the specific intersection approaches or system elements experiencing LOS F conditions.

Separate LOS Reporting by Mode and System Element

LOS is reported separately for each mode operating on a given system element [although some other modes, such as large trucks, recreational vehicles (RVs), and motorcycles are currently considered members of the automobile model for HCM analysis purposes]. Each mode's travelers have different perspectives and potentially experience very different conditions while traveling along a particular roadway. Using a blended LOS risks overlooking quality of service deficiencies for nonautomobile travelers that discourage the use of those modes, particularly if the blended LOS is weighted by the number of modal travelers. Other measures, such as person-delay, can be used when an analysis requires a combined measure.

LOS is reported separately, by mode, for a given system element.

Identical values of some service measures (e.g., delay) can produce different LOS results, depending on the system element to which the service measure is applied. The TRB Committee on Highway Capacity and Quality of Service (HCQS Committee) believes that travelers' expectation of performance varies at different system elements (e.g., unsignalized intersections versus signalized intersections) but realizes that further research is needed to understand fully the variation in traveler perceptions of LOS across facility types.

LOS as Part of a Bigger Picture

Neither LOS nor any other single performance measure tells the full story of roadway performance. Depending on the particulars of a given analysis, queue lengths, demand-to-capacity ratios, average travel speeds, indicators of safety, quantities of persons and vehicles served, and other performance measures may be just as or even more important to consider, whether or not they are specifically called out in an agency standard. For this reason, the HCM provides methods for estimating a variety of useful roadway operations performance measures, and not just methods for determining LOS. Chapter 7, Interpreting

No single performance measure tells the full story of roadway performance.

HCM and Alternative Tool Results, lists the major performance measures available from each chapter of Volumes 2 and 3.

Duration of an operating condition can be important to know, since it helps describe the severity of the condition (e.g., the duration of a LOS F condition). In cases where demand exceeds capacity, duration *must* be known in order to set the analysis period long enough so that all demand is served and all relevant performance measures can be calculated properly. Frequency and probability of a particular condition occurring (e.g., likelihood or frequency of queue storage being exceeded during an analysis period) are also useful descriptors.

4. SERVICE MEASURES

DEFINITION AND CHARACTERISTICS

Service measures reflect traveler perceptions and can be influenced by agency actions.

Service measures are performance measures used to define LOS for transportation system elements.

Ideally, service measures should exhibit the following characteristics:

- Service measures should reflect travelers' perceptions (i.e., measures should reflect things travelers can perceive during their journey);
- Service measures should be useful to operating agencies (e.g., agency actions should be able to influence future LOS);
- Service measures should be directly measurable in the field (e.g., an analyst wishing to determine LOS for a two-lane highway used for recreational access can go into the field and directly measure average travel speed); and
- Service measures should be estimable given a set of known or forecast conditions (e.g., a method is provided in Chapter 15 to estimate the average travel speed for a two-lane highway, given inputs for roadway and traffic conditions).

SERVICE MEASURE SELECTION

Service measure selection.

Historically, the selection of a service measure or measures for an analysis methodology has been based on the collective opinion and judgment of TRB's HCQS Committee. The service measure threshold values that identify the breakpoints between each level of service have also been determined by the HCQS Committee. This approach was necessary, because until recently little information was available on the subject of how travelers evaluate operating conditions. However, it has been the intent of the committee to select service measures that it felt would be highly correlated with travelers' personal assessments of the operating conditions. Since 1993, there has been considerably more research focused on determining appropriate service measures based directly on traveler input. While this area of research was still immature at the time of publication of this edition of the HCM, the HCQS Committee intends to monitor and evaluate future research in this area for potential inclusion in subsequent editions of the HCM.

Studies that seek to determine service measures and thresholds based on traveler perceptions obviously use research approaches that directly involve a sample of travelers. Some of the methods used to obtain this direct traveler input include in-field experiments (e.g., driving or riding courses), simulated in-field experiments (such as through the use of video presentations), focus groups, and surveys. The study participants are typically asked to rate the conditions they are presented with on a scale of "very good" to "very poor," or something similar. The qualitative ratings are later converted to numeric values for analysis purposes. Some challenges to these types of studies include designing the instrument (field experiment, focus group, etc.) to capture all of the roadway, traffic, and control factors that might affect travelers' perceptions of operating

conditions; excluding factors that may not be relevant but could distract study subjects; recruiting an adequate sample of study participants from both a quantity and a diversity perspective; replicating desired conditions (for in-field experiments) for repeated observations; and accounting for the distribution of LOS responses that will result from each test scenario in the analysis methodology.

The advantage of this type of research approach is that, with application of an appropriate analysis methodology, multiple variables can be considered simultaneously, consistent with the high likelihood that travelers consider multiple factors when they evaluate the operating conditions. Including multiple factors also gives agencies more options in seeking to achieve a desired LOS for a given mode or in balancing the needs of various modes. All variables found to be statistically significant are incorporated into a mathematical function (hereinafter referred to as a *model*). In the model, the coefficients (i.e., weighting factors) associated with each of the variables are determined directly through the statistical analysis. The output from such a model is a value often referred to as a LOS score. The LOS score value generally represents the average score that travelers would give a facility or service. Furthermore, some of the methodologies can directly estimate the threshold values that are used to distinguish between the LOS categories, again, on the basis of traveler input. To determine the LOS, the LOS score value is compared with the statistically estimated threshold values.

While variables representing any number of things can be included in this type of model, for models to be useful from a practical perspective, only variables representing operational or design conditions are usually included. Operational conditions refer to variables such as delay and speed (i.e., performance measures), while design conditions refer to variables such as median type and sidewalk presence. Traveler characteristics (e.g., age, gender, income) can certainly affect LOS perceptions; however, these data are difficult to collect in a transportation engineering situation. Thus, their utility in a LOS model is limited.

Several methodological approaches have been applied to relate traveler perceptions directly to LOS, including regression-based methods (*1–4*), ordered probit models (*5*, *6*), and fuzzy clustering (*7*). These studies have addressed facilities such as urban and rural freeways, arterial streets, and signalized intersections. LOS methods resulting from some of these studies have been included in the HCM 2010, while others have been studied by the HCQS Committee to improve the understanding of techniques used in estimating traveler-based LOS.

This edition of the HCM is the first to incorporate LOS methodologies that are based directly on results from traveler perceptions of LOS. As the field of traveler perception of LOS research continues to mature and results from regional studies are validated nationally, the HCQS Committee expects to continue to include these new LOS methodologies in future editions of the HCM. When research is not available to support traveler-perceived LOS methodologies,

HCQS Committee–selected service measures and thresholds continue to be used in this edition of the HCM.

DETERMINATION OF LOS F

The threshold between LOS E and LOS F is based on the judgment of the HCQS Committee in some instances and is determined directly from research on traveler perceptions of LOS in others. One example of where the service measure and LOS thresholds were determined by the HCQS Committee is basic freeway segments; density was selected as the service measure and the LOS E–F density threshold value was selected as the density at which traffic flow transitions from uncongested to congested. Bicycling on urban streets is an example of where the service measures were determined from traveler perception of LOS research; the LOS E–F threshold was chosen as a value that represents the transition to a totally unacceptable condition (i.e., an average rider will not ride under these conditions). Thresholds between LOS A and E may be based on ranges of values that define particular operating conditions or may simply provide an even gradation of values from LOS A to E. As mentioned previously, in some studies on traveler perceptions of LOS, the methodological approach explicitly yields the model variables (e.g., speed, median presence) as well as the specific LOS thresholds. However, these thresholds are still a function of the total number of LOS categories originally included in the study.

The v/c ratio is often used to define the LOS E–F threshold, but not other LOS thresholds.

The volume-to-capacity (v/c) ratio (or, more correctly, the demand-to-capacity, d/c, ratio) is a special-case service measure. It cannot be directly measured in the field, nor is it a measure of traveler perceptions. Until capacity is reached (i.e., when flow breaks down on uninterrupted-flow facilities and when queues build on interrupted- or uninterrupted-flow facilities), the v/c ratio is not perceivable by travelers. Therefore, the HCM often uses the v/c ratio to define the LOS E–F threshold, but not to define other LOS thresholds.

SERVICE MEASURES FOR SPECIFIC SYSTEM ELEMENTS

Cross-Cutting Issues

Automobile Mode

LOS for the automobile mode is reflective of automobile driver and passenger perspectives, but not necessarily those of other motorized vehicle users.

A facility's capacity to serve the automobile mode reflects the effects of all motorized vehicles using the facility, including trucks, RVs, motorcycles, and intercity buses. In contrast, LOS for the automobile mode reflects the perspective of automobile drivers and passengers, but not necessarily the perspectives of other motorized vehicle users. Although automobiles are usually the dominant motorized vehicle type on roadways, analysts should use care in interpreting LOS results in special cases, such as intermodal-terminal access routes, where trucks may dominate.

Pedestrian and Bicycle Modes

Depending on local regulations, pedestrians and bicycles may be allowed on all types of uninterrupted-flow facilities, including sections of freeways. However, research is only available to support LOS estimation methods for bicyclists traveling on two-lane and multilane highways. Pathways that are

Pathways parallel to freeways and multilane highways are analyzed by using the off-street facility procedures.

parallel to freeways and multilane highways use the service measures for off-street pedestrian and bicycle facilities. Of the various types of interrupted-flow system elements, pedestrian and bicycle service measures are provided for urban street facilities, urban street segments, signalized intersections, and off-street pedestrian and bicycle facilities. Pedestrian LOS can also be calculated for two-way STOP-controlled intersections and roundabouts.

Transit Mode

Transit service measures are provided only for urban streets. Consult the TCQSM for service measures for other situations.

Bus service on uninterrupted-flow facilities typically serves longer-distance trips, with few (if any) stops. The transit service measures provided in the *Transit Capacity and Quality of Service Manual* (TCQSM) (*8*) can be used to evaluate bus service along uninterrupted-flow facilities, as well as rail service operating within an uninterrupted-flow facility's right-of-way.

The HCM provides transit service measures for urban street facilities and segments to facilitate multimodal comparisons of urban-street LOS. The TCQSM's service measures can be used to evaluate transit LOS at specific bus stops, for specific routes, for specific origin–destination pairs, and for specific areas. Some of the HCM's performance measures, such as delay, may also be useful in multimodal comparisons—for example, in comparing person-delay at an intersection with and without a transit preferential treatment, such as a queue-jump lane.

Freeways and Multilane Highways

Automobile Mode

Density is the automobile service measure for all freeway and multilane highway system elements.

Although travel speed is a major concern of drivers that relates to service quality, freedom to maneuver within the traffic stream and proximity to other vehicles are equally noticeable concerns. These qualities are related to the *density* of the traffic stream. Unlike speed, density increases as flow increases up to capacity, resulting in a service measure that is both perceivable by motorists and is sensitive to a broad range of flows. Density is used as the service measure for freeway facilities, basic freeway segments, ramp junctions, weaving segments, and multilane highways.

Bicycle Mode

Bicycle LOS for multilane highways is based on a *bicycle LOS score* model. The model uses variables determined from research that relate to bicyclists' comfort and perceived exposure while riding on multilane highways, such as separation from traffic, motorized traffic volumes and speeds, heavy-vehicle percentage, and pavement quality.

Higher vehicle volumes, a greater proportion of trucks and buses, and higher vehicle speeds all act to decrease a bicyclist's perceived comfort and traffic exposure. Striped bicycle lanes or roadway shoulders add to the perceived sense of traffic separation and improve the LOS. Pavement quality affects bicyclists' ride comfort: the better the pavement quality, the better the LOS.

Two-Lane-Highway Measures

Automobile Mode

Traveler expectations for and travel conditions on two-lane highways are different from those for other uninterrupted-flow facilities.

Traffic operations on two-lane, two-way highways differ from those on other uninterrupted-flow facilities. Lane changing and passing are possible only in the face of oncoming traffic. Passing demand increases rapidly as traffic volumes increase, and passing capacity in the opposing lane declines as volumes increase. Therefore, on two-lane highways, unlike other types of uninterrupted-flow facilities, normal traffic flow in one direction influences flow in the other direction. Motorists must adjust their travel speeds as volume increases and the ability to pass declines.

Efficient mobility is the principal function of major two-lane highways that connect major traffic generators or that serve as primary links in state and national highway networks. These routes tend to serve long-distance commercial and recreational travelers, and long sections may pass through rural areas without traffic-control interruptions. Consistent high-speed operations and infrequent passing delays are desirable for these facilities.

Other paved, two-lane rural highways are intended to serve primarily an accessibility function. Although beneficial, high speed is not the principal concern. Delay—as indicated by the formation of platoons—is more relevant as a measure of service quality.

Two-lane roads also serve scenic and recreational areas in which the vista and environment are meant to be experienced and enjoyed without traffic interruption or delay. A safe roadway is desired, but high-speed operation is neither expected nor desired. For these reasons, three service measures are used for two-lane highways: *percent time-spent-following, average travel speed,* and *percent free-flow speed.*

Percent time-spent-following defined.

Percent time-spent-following represents the freedom to maneuver and the comfort and convenience of travel. It is the average percentage of travel time that vehicles must travel in platoons behind slower vehicles because of the inability to pass. Percent time-spent-following is difficult to measure in the field. However, the percentage of vehicles traveling with headways of less than 3 s at a representative location can be used as a surrogate measure.

Average travel speed defined.

Average travel speed reflects the mobility on a two-lane highway: it is the length of the highway segment divided by the average travel time of all vehicles traversing the segment in both directions during a designated interval.

Percent of free-flow speed defined.

Percent of free-flow speed represents the ability of vehicles to travel at or near the posted speed limit.

LOS criteria use one or two of these measures. On major two-lane highways, for which efficient mobility is paramount, both percent time-spent-following and average travel speed define LOS. However, roadway alignments with reduced design speeds will limit the LOS that can be achieved. On highways for which accessibility is paramount and mobility less critical, LOS is defined only in terms of percent time-spent-following, without consideration of average travel speed. On two-lane highways in developed rural areas, LOS is defined in terms of percent of free-flow speed.

Bicycle Mode

Bicycle LOS for two-lane highways is determined by a *bicycle LOS score* model in the same manner as described above for multilane highways, with the addition of on-highway parking as a factor that may exist on two-lane highways. The presence of on-highway parking negatively affects bicycle LOS because bicyclists tend to leave a buffer between themselves and parked vehicles, resulting in less separation between themselves and moving vehicles.

Urban Street Facility and Segment Measures

Automobile Mode

The service measure for the automobile mode on an urban street is the percent of base free-flow speed. Motorists traveling along arterial streets expect to be able to travel at or near the posted speed limit between intersections and to have to stop only infrequently. As delay due to traffic control devices and to other roadway users (e.g., vehicles stopped in a travel lane waiting to turn, buses stopping to serve passengers, or pedestrian crossings) increases, the lower the average speed and the lower the perceived LOS.

Research on automobile travelers' perceptions of LOS, as part of the National Cooperative Highway Research Program (NCHRP) 3-70 project, revealed that a combination of stops per mile and left-turn lane presence at signalized intersections had the highest statistical significance for predicting automobile LOS. However, the HCQS Committee elected to retain usage of a time-based service measure to analyze automobile LOS on urban streets for this edition of the HCM. The alternative NCHRP 3-70 methodology is also presented in Chapter 17, Urban Street Segments, since it is well suited for applications with a focus on determining multimodal LOS trade-offs and designing complete streets.

Pedestrian Mode

Urban street pedestrian LOS combines the quality of walking along a street, crossing at signalized intersections, and crossing the street between traffic signals.

Pedestrian LOS for urban streets is based on a *pedestrian LOS score* model that includes variables determined from research on pedestrians' perceptions of LOS. These variables relate to pedestrians' experiences walking along street segments between signalized intersections, crossing side streets at signalized intersections, and crossing the street between signalized intersections.

The segment component relates to both the density of pedestrians along the street and to pedestrian comfort and perceived exposure to traffic. The pedestrian-density indicator is a function of pedestrian volumes and sidewalk width, while the nondensity indicator is a function of separation from traffic due to distance and physical objects, sidewalk presence and width, and motorized traffic volumes and speeds. The worse of the two indicators is used to determine pedestrian-perceived segment LOS. The nondensity indicator is predominately used in suburban and rural settings.

The signalized-intersection component relates to pedestrian delay and perceived exposure to or interaction with traffic. The exposure elements of the indicator include potentially conflicting traffic volumes, parallel traffic volumes, parallel traffic speed, crossing width, and channelizing-island presence.

The roadway-crossing component is a function of the lesser of the delay in waiting for a gap to cross the street and the delay involved in diverting to the nearest signalized intersection. It also incorporates the segment and signalized-intersection components, which relate to the quality of the pedestrian environment experienced when pedestrians divert to a signal, either because of lower delay or a prohibition on crossings between signalized intersections.

Overall, pedestrian LOS is improved by the provision of sidewalks, wider sidewalks, a greater degree of separation from traffic, and reduced delays crossing the street at both signalized and unsignalized locations. Higher traffic volumes, higher traffic speeds, and wider streets all tend to reduce pedestrian LOS.

Bicycle Mode

Urban street bicycle LOS combines the quality of bicycling along the street between traffic signals and the quality of passing through signalized intersections.

Bicycle LOS for urban streets is based on a *bicycle LOS score* model that includes variables determined from research on bicycle riders' perceptions of LOS. These variables relate to bicyclists' experiences at signalized intersections and their experiences on street segments between signalized intersections. The intersection component relates to bicyclist comfort and perceived exposure to traffic and is a function of separation from traffic, cross-street width, and motorized traffic volumes. The segment component similarly relates to comfort and perceived exposure. It is a function of separation from traffic, motorized traffic volumes, traffic speeds, heavy-vehicle percentage, presence of parking, and pavement quality. The frequency of unsignalized intersections and driveways between traffic signals is also a factor in the LOS score value.

Higher vehicle volumes, a greater proportion of trucks and buses, higher vehicle speeds, and presence of parking all act to decrease a bicyclist's perceived comfort and traffic exposure. Striped bicycle lanes or roadway shoulders add to the perceived sense of traffic separation and improve the LOS. Pavement quality affects bicyclists' ride comfort: the better the pavement quality, the better the LOS.

Transit Mode

The transit service measure applies to bus, streetcar, and at-grade light rail services that make stops along an urban street.

The service measure combines traveler perceptions walking to a transit stop, waiting for a transit vehicle, and riding on the vehicle.

Transit LOS for urban streets is based on a *transit LOS score* model that includes variables determined from research on transit riders' perceptions of LOS. These variables relate to passengers' experiences walking to a transit stop on the street, waiting for the transit vehicle, and riding on the transit vehicle. The walking-to-the-stop component is based on the street's pedestrian LOS score: transit passengers are usually pedestrians before and after their transit trip—and improvements to the pedestrian environment along streets with transit service contribute to a better LOS. The waiting component is a function of the transit vehicle frequency (relating to wait time and trip-making convenience), service reliability (unplanned passenger waiting time at the stop), and the presence of shelters and benches (which make waiting time more comfortable). Finally, the riding-on-the-vehicle satisfaction is a function of average travel speed (a convenience factor) and passenger loads (a comfort factor).

Control delay is the automobile service measure for urban street intersections.

Urban Street Intersections

Automobile Mode

The service measure for the automobile mode at all urban-street intersections—including signalized intersections, all-way STOP-controlled intersections, two-way STOP-controlled intersections, roundabouts, and interchange ramp terminals—is *control delay.*

Control delay, which was defined in Chapter 4, Traffic Flow and Capacity Concepts, is a measure of driver discomfort, frustration, fuel consumption, and increased travel time. It depends on a number of variables, which are different depending on whether the intersection is signalized or unsignalized. As control delay increases, LOS worsens. The maximum control delay allowed for a given LOS at unsignalized intersections is lower than for signalized intersections due to differing driver expectations. In the HCQS Committee's opinion, the expectation of drivers is that a signalized intersection is designed to carry higher traffic volumes and experience greater delay than an unsignalized intersection.

Pedestrian, Bicycle, and Transit Modes

At the time of publication, there was insufficient research to be able to provide pedestrian and bicycle LOS for urban street intersections, except for signalized intersections and—for pedestrians only—two-way STOP-controlled intersections. The HCM provides transit LOS measures only at the urban street segment and facility levels.

Signalized Intersections

Pedestrian LOS at signalized intersections is based on a *pedestrian LOS score* model that incorporates conflicting motorized vehicle volumes and speeds, crosswalk length, average pedestrian delay, and the presence of right-turn channelizing islands. Pedestrian LOS improves with lower motorized vehicle volumes and speeds, shorter crosswalk lengths, lower delay, and the provision of right-turn channelizing islands.

Bicycle LOS at signalized intersections is based on a *bicycle LOS score* model that incorporates perceived separation from motorized vehicle traffic, motorized vehicle volumes, cross-street width, and presence and utilization of on-street parking. Bicycle LOS improves with greater perceived separation from motorized vehicle traffic, lower motorized vehicle volumes, shorter cross-street widths, and reduced on-street parking conflicts.

Two-Way STOP-Controlled Intersections

Pedestrian LOS at two-way STOP-controlled intersections is based on average pedestrian *control delay* crossing the major street. Lower vehicle volumes, presence of a median, and provision of pedestrian crossing treatments that improve motorist yielding rates all help to improve pedestrian LOS.

Off-Street Pedestrian and Bicycle Facilities

Pedestrian Mode

The service measure for off-street exclusive pedestrian facilities is pedestrian space.

Off-street facilities used exclusively by pedestrians (e.g., pedestrian pathways, plazas, and stairways) use *pedestrian space* as the service measure. As the space available to pedestrians increases, the ability to move at one's desired speed along one's desired line of travel increases. Therefore, as the space available to a pedestrian on an off-street facility increases, so does the LOS.

The pedestrian service measure for shared off-street facilities is the number of times per hour a pedestrian meets or is passed by bicyclists.

When an off-street facility is shared by pedestrians and bicycles, pedestrian quality of service is much more affected by the bicyclists using the facility than by other pedestrians because of the speed differential between the two types of travelers. Therefore, the pedestrian service measure for shared off-street facilities is based on *events,* the number of times per hour that a pedestrian is met by or passed by bicyclists. The greater the number of bicyclists on a shared facility, the lower the pedestrian LOS.

Bicycle Mode

The bicycle service measure for all off-street facilities is a bicycle LOS score.

Bicycle LOS for off-street bicycle facilities, both exclusive and shared, is based on a *bicycle LOS score* model that includes variables determined from research on bicycle riders' perceptions of LOS. These variables consist of the number of times a bicyclist meets other path users per minute, the number of times per minute on average that a bicyclist passes or is delayed in passing other path users, the presence of a centerline, and the path width. As the number of other path users (including bicyclists) increases, the LOS declines. Wider paths and the presence of a centerline contribute to better LOS.

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CHAPTER 6
HCM AND ALTERNATIVE ANALYSIS TOOLS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

It is well recognized that the models of the *Highway Capacity Manual* (HCM) are part of a continuum available to the analyst for highway capacity and quality-of-service analyses. For example, transportation planners and engineers have different needs in scope and level of detail. Planners focus on network performance in broad, general terms to understand supply–demand interactions between network capacity and flows. Of primary interest are the interactions of land uses and the transportation system. Engineers, in contrast, need to know how changes in the design of a specific system element or changes in the way it operates will affect its performance in terms of capacity, delays, queuing characteristics, and other measures. As a result, different traffic analysis tools are available to serve different analysis needs.

Chapter 6, HCM and Alternative Analysis Tools, begins by describing the HCM-based tools available to the analyst. In an *operations-level analysis*, which is one such tool, HCM methodologies are applied directly and the user supplies all required inputs to the procedure. Many planning, preliminary engineering, and design applications do not require the level of accuracy provided by an operations analysis and substitute *default values* for some (or nearly all) of a methodology's inputs. *Generalized service volume tables* are sketch-planning tools that provide an estimate of the maximum volume a system element can carry at a given level of service (LOS), given a default set of assumptions about the system element. The use of local default values and local generalized service volume tables helps reduce the uncertainty in the results of analyses that use these tools, compared with using the HCM's national default values and tables. The chapter's two appendices provide guidance on developing local default values (Appendix A) and generating local generalized service volume tables (Appendix B).

Analysts might consider alternative tools as a supplement to HCM methods for a variety of reasons. Alternative tools might be considered in the following analyses, among others: those outside the range covered by an HCM methodology, those requiring performance measures not produced by the HCM, and those where the quantity of data required makes HCM methods impractical. To assist analysts in evaluating and applying alternative tools, this chapter describes traffic modeling concepts and terminology, identifies conceptual differences between analytical and simulation tools, provides a framework for applying alternative tools, and lists criteria to consider in the selection of a traffic analysis tool.

2. HCM-BASED TOOLS

GENERALIZED SERVICE VOLUME TABLES

Service volume tables provide estimates of the maximum number of vehicles a system element can carry at a particular LOS, given a set of assumed conditions.

A service volume table provides an analyst with an estimate of the maximum number of vehicles that a system element can carry at a given LOS. The use of a service volume table is most appropriate in certain planning applications in which evaluation of every segment or node within a study area is not feasible. Examples of these applications are city, county, or statewide planning studies where the size of the study area makes a capacity or LOS analysis for every system element infeasible. For these types of planning applications, the focus of the effort is to highlight potential problem areas (for example, locations where demand may exceed capacity or where a desired LOS threshold may be exceeded). For such applications, a service volume table can be a useful sketch-planning tool, provided the analyst understands the limitations of this method. Once potential problem areas have been identified, other tools (HCM-based or alternative) can be used to perform more detailed analyses for locations of interest.

As described in more detail in Appendix B, generalized service volume tables are developed by holding constant all input values to a particular HCM methodology—except demand volume. Demand volume is increased until the measure of effectiveness (MOE) for the methodology reaches the threshold for a given LOS (e.g., the threshold between LOS B and C). That demand volume then becomes the *service volume* for the given LOS (in the example above, for LOS B). The service volume represents the maximum number of vehicles that the system element can carry at the given LOS, given the assumed inputs.

Service volume results should be applied with care, since actual conditions will likely vary in some way from the assumptions used to develop the table.

The characteristics of any given system element will likely vary in some way from the assumed input values used to develop a service volume table. Therefore, the results from a service volume table should be treated as rough approximations. Service volume tables should not be used as a substitute for other tools when a final determination of the operational adequacy of a particular roadway is made.

The assumptions built into a table may be average (or typical) values or conservative values. The choice affects how results from the table are interpreted.

Service volumes can be sensitive to the choice of input values, depending on the particular methodology and input parameter.

For ease of use, generalized service volume tables require a minimum of user inputs—typically, key design parameters that have the greatest influence on a facility's capacity and LOS, such as the number of lanes. Given these inputs, a user can then read the service volume for a given LOS directly from the table and compare it with the actual or forecast volume for a system element. A volume greater than the service volume for the desired LOS indicates the need for further analysis. Depending on the assumptions used to develop the table (i.e., average or typical values versus conservative values) and the sensitivity of the service volumes to the default values used, volumes somewhat less than the identified service volume (as much as 25% below the service volume in some cases) could also suggest the need for further analysis.

APPLICATION OF DEFAULT VALUES TO HCM METHODOLOGIES

Many planning, preliminary engineering, and design analyses apply HCM methodologies directly but use default values for some or many of the input parameters.

In most planning, preliminary engineering, and design applications of the HCM, an analyst applies an HCM methodology taken directly from one or more of the chapters in Volumes 2 and 3 but uses default values for some (or many) of the input parameters. These types of analyses are frequently used to evaluate or design for future operations. Therefore, not all of the input parameters required by an HCM methodology (e.g., heavy-vehicle percentage or peak hour factor) may be available or readily forecast. Default values can also be applied when current operations are evaluated as part of a screening effort, similar to the way service volume tables are applied. Because users have control over which input parameters are defaulted, the uncertainty of the analysis results is reduced compared with the application of a service volume table, which potentially reduces the number of problem areas flagged for further analysis.

Although the HCM provides default values for a number of methodologies, the analyst should be mindful that these values represent typical national values and that typical conditions within a state, region, or community may be different. When default values occur frequently in analyses, the use of local default values can help further reduce the uncertainty in the analysis results. Appendix A provides guidance on developing local default values.

OPERATIONS-LEVEL HCM ANALYSIS

Operations-level HCM analyses apply the HCM methodology directly and use no, or minimal, default values.

In an operations-level analysis, an analyst applies an HCM methodology directly from one or more of the chapters in Volumes 2 and 3 and supplies all of the required input parameters from measured or forecast values. No, or minimal, default values are used. Therefore, compared with generalized service volume tables and the application of default values, operations-level analyses provide the highest level of accuracy. However, as discussed in Chapter 7, Interpreting HCM and Alternative Tool Results, analysts must still account for variability, uncertainty, and measurement errors in input data, and their impact on the analysis outputs, in the interpretation of analysis results.

The chapters in Volumes 2 and 3 identify methodological limitations and special cases where analysts may wish to consider alternative tools.

An operations-level analysis is applicable to any situation covered by an HCM methodology. Analysts should refer to the Limitations of the Methodology and Special Cases sections of the HCM chapter that covers the system element being evaluated to ensure that the HCM methodology is appropriate for the specifics of the situation being studied. A situation that is outside the range of conditions covered by the methodology would be one reason for considering an alternative tool. Other cases when alternative tools might be considered are discussed in the next section.

3. ALTERNATIVE TOOLS

OVERVIEW

Alternative tools include all non-HCM procedures that may be used to evaluate highway system performance.

Alternative tools include all analysis procedures outside of the HCM that may be used to compute measures of transportation system performance for analysis and decision support. Most alternative tools take the form of software products, and there is an abundance of literature describing alternative tools and their applications. The purpose of this section is to categorize the most commonly used tools, to identify the conditions under which they might be used to supplement the HCM procedures, and to suggest a framework and guidelines for their application that will maximize their compatibility with the HCM deterministic procedures. The intent is not to identify or compare specific tools or to duplicate the wealth of literature that exists on the general subject of traffic performance analysis.

This section describes the types of available tools (which are generally software products) but does not identify or compare specific tools.

Clearly, readers should use the material presented here in conjunction with other documents that also address analysis, modeling, and simulation of transportation systems. A considerable volume of authoritative information on this subject is available in the Federal Highway Administration's *Traffic Analysis Toolbox*. Four volumes of the *Toolbox* provide general guidance on the use of traffic analysis tools, including the HCM:

A useful reference on traffic operations modeling is the Federal Highway Administration's Traffic Analysis Toolbox.

- *Volume I: Traffic Analysis Tools Primer* (*1*) presents a high-level overview of the different types of traffic analysis tools and their role in transportation analyses.
- *Volume II: Decision Support Methodology for Selecting Traffic Analysis Tools* (*2*) identifies key criteria and circumstances to consider in selecting the most appropriate type of traffic analysis tool for the analysis at hand.
- *Volume III: Guidelines for Applying Traffic Microsimulation Modeling Software* (*3*) provides a recommended process for using traffic microsimulation software in traffic analyses.
- *Volume VI: Definition, Interpretation, and Calculation of Traffic Analysis Tools Measures of Effectiveness* (*4*) describes the appropriate definition, interpretation, and computation of MOEs for traffic operations and capacity improvements. This report provides information and guidance on which MOEs should be produced, how they should be interpreted, and how they are defined and calculated in traffic analysis tools. The document recommends a basic set of MOEs that can help rapidly assess the current problems and the benefits of alternative improvements at the system level, in a form readily understandable by the decision maker.

Other volumes of the *Toolbox* deal with the application of alternative tools for specific application scenarios.

The material in this section focuses primarily on the automobile mode. The importance of other modes—particularly bicycles, pedestrians, and transit—is recognized; however, experience with the use of alternative tools to address these modes is limited. Some alternative tools address nonautomobile modes

explicitly and some do not. The *Traffic Analysis Toolbox* (*1–3*) mentions the inability to deal with “interferences that can occur between bicycles, pedestrians, and vehicles sharing the same roadway” as a limitation of most simulation tools. Where appropriate, specific guidance is presented on the treatment of bicycles, pedestrians, and buses in the chapters of Volumes 2 and 3. The best source of guidance for the use of alternative tools for other modes is the detailed documentation and user’s guide provided with tools that offer these features.

TRAFFIC MODELING CONCEPTS AND TERMINOLOGY

Hierarchy of Modeling Terminology

Modeling terminology has not been applied consistently throughout the realm of traffic analysis tools. For purposes of the HCM, the following terminology will be used to distinguish between different objects and processes that have been referred to in the literature simply as a “model”:

- An *algorithm* is, by dictionary definition (*5*), “a set of rules for solving a problem in a finite number of steps.” This definition suits the HCM’s purposes.
- A *model* is, by dictionary definition (*5*), “a hypothetical description of a complex entity or process.” Here is the root of the inconsistent usage. By this definition the word can be, and has been, applied to many different objects. A more focused definition is required. One definition in common use is that a model is “a representation of a system that allows for investigation of the properties of the system and, in some cases, prediction of future outcomes”(*6*). For HCM purposes, the term will be used in this sense but will be more precisely defined as “a procedure that uses one or more algorithms to produce a set of numerical outputs describing the operation of a highway segment or system, given a set of numerical inputs.” By this definition, each of the performance analysis procedures specified in Volumes 2 and 3 constitutes a model. This term will generally be used with an adjective to denote its purpose (e.g., delay model).
- A *computational engine* is the software implementation of one or more models that produces specific outputs given a set of input data.
- A *traffic analysis tool,* often abbreviated in the HCM as a “tool,” is a software product that includes, at a minimum, a computational engine and a user interface. The purpose of the user interface is to facilitate the process of entering the input data and interpreting the results.
- A *model application*, sometimes referred to as a *scenario,* specifies the physical configuration and operational conditions to which a traffic analysis tool is applied.

Inconsistency in terminology arises because each of these five objects has been characterized as a model in the literature, since each one satisfies the dictionary definition. The distinction between the five terms is made here in the hope of promoting more consistent usage.

Additional Modeling Definitions

Another set of terminology that requires more precise definitions deals with the process by which the analyst ensures that the modeling results provide a realistic representation of the real world. The following terms are defined elsewhere (*3*):

- *Verification:* The process by which the software developer and other researchers check the accuracy of the software implementation of traffic operations theory. The extent to which a given tool has been verified is listed as an important tool selection criterion in this chapter.
- *Calibration*: The process by which the analyst selects the model parameters that cause the model to best reproduce field-measured local traffic conditions.
- *Validation:* The process by which the analyst checks the overall model-predicted traffic performance for a street–road system against field measurements of traffic performance, such as traffic volumes, travel times, average speeds, and average delays. Model validation is performed on the basis of field data not used in the calibration process.

Traffic Analysis Tool and Model Categories

The *Traffic Analysis Toolbox* identifies the following categories of traffic analysis models:

- *Sketch-planning tools* produce general order-of-magnitude estimates of travel demand and transportation system performance under different transportation system improvement alternatives.
- *Travel demand models* forecast long-term future travel demand on the basis of current conditions and projections of socioeconomic characteristics and changes in transportation system design.
- *HCM-based analytical deterministic tools* quickly predict capacity, density, speed, delay, and queuing on a variety of transportation facilities.
- *Traffic signal optimization tools* are primarily designed to develop optimal signal phasing and timing plans for isolated signalized intersections, arterial streets, or signal networks.
- *Macroscopic simulation models* are based on the deterministic relationships of the flow, speed, and density of the traffic stream.
- *Microscopic simulation models* simulate the movement of individual vehicles on the basis of car-following and lane-changing theories.
- *Mesoscopic models* combine the properties of microscopic and macroscopic simulation models.
- *Hybrid models* employ microscopic and mesoscopic models simultaneously. These tools are intended to be applied to very large networks containing critical subnetworks connected by several miles of essentially rural facilities. Microscopic modeling is applied to the critical subnetworks, while the connecting facilities are modeled at the

Hybrid models, used with very large networks, apply microscopic modeling to critical subnetworks and mesoscopic or macroscopic modeling to the connecting facilities.

mesoscopic or macroscopic level. Regional evacuation models are a typical example of hybrid model application.

Stochastic and Deterministic Models

A *deterministic* model is not subject to randomness. Each model run will produce the same outcome. If these statements are not true and some attribute of the model is not known with certainty, the model is *stochastic*. Random variables will be used to represent those attributes of the model not known with certainty. Descriptions of how these random numbers are selected to obtain sample values of the parameter of interest (i.e., from its cumulative distribution function) can be found in various texts (e.g., *7–10*). Different random number sequences will produce different model results; therefore, the outcome from a simulation tool based on a stochastic model cannot be predicted with certainty before analysis begins. Stochastic models aid the user in incorporating variability and uncertainty into the analysis.

The HCM's methodologies are deterministic—given the same set of inputs, the methods will produce the same result each time.

Most simulation models are stochastic—given identical inputs but a different random number seed, model runs will produce different results.

Static Flow and Time-Varying Flow Models

The terms *static flow* and *time-varying flow* relate to the temporal characteristics of the traffic flows in the simulation model. Basically, the terms differentiate between a model that uses constant traffic flow rates from one time period to another and a model that does not. This differentiation is not to be confused with whether the model can represent internally time-varying flows that occur because of simulated events (e.g., incidents, signal cycling, ramp metering, high-occupancy-vehicle lane closures). The difference is in the type of input flows that can be specified.

In the static flow case, traffic flows are provided just once, as a set of constants. A tool may vary the individual headways stochastically, but the flow rates are fixed. Put another way, the origin–destination (O-D) matrix is fixed and does not change throughout the duration of the analysis.

In the time-varying case, flow rates can change with time. More than one set of flow rates must be specified so that the O-D matrix can vary over time. The flexibility of specifying more than one set of flow rates is particularly useful when major surges in traffic need to be examined, such as the ending of a special event or peak periods when a pronounced variation in traffic flows exists.

In static-flow models, users provide a single set of flow rates. The model may vary headways, but the O-D matrix is fixed and does not change throughout the duration of the analysis.

Time-varying models allow flow rates to change with time. Users supply more than one set of flow rates so that the O-D matrix can vary over time. Most models change flows once an hour, but some allow more frequent changes.

Descriptive and Normative Models

The terms *descriptive* and *normative* refer to the objective of performing the analysis with simulation models. If the objective of the model is to describe how traffic will behave in a given situation, the model is most likely to be descriptive. It will not try to identify a given set of parameters that provide the best system performance but rather will show how events will unfold given a logic that describes how the objects involved will behave. For example, a simulation model could predict how drivers will behave in response to traffic flow conditions. A model attempting to shape that behavior by trying to force drivers to maintain specific headways would not be a descriptive model.

Normative models try to identify a set of parameters providing the best system performance. An external influence (most often referred to as an objective

Descriptive models show how events unfold given a logic that describes how the objects involved will behave.

Normative models try to identify a set of parameters that provide the best system performance.

function) tries to force the system to behave in some optimal way. A good example is a model that tries to optimize signal timings. Another illustration is a freeway network model that requires drivers to alter their path choices to optimize some measure of system performance. In both cases, the behavior of the system is modified through an external influence, probably on an iterative basis, to create a sequence of realizations in which the objective function value is improved, as in minimizing total travel time or total system delay.

The distinction can be restated this way: if the model has an objective and seeks to optimize that objective, it is a normative model. Conversely, if it has an objective but does not seek to optimize that objective by changing the design or operational parameters (e.g., signal timing), it is a descriptive model.

Traffic assignment models are a special case, because they are based on an objective that is gradually improved over a sequence of iterations. In this case, the objective is for each driver to minimize either the travel time for the trip or some other quantitative measure of the general cost or disutility associated with the trip. Traffic assignment models are characterized as either *static* or *dynamic,* depending on whether the O-D characteristics are constant or time-varying. Most simulation tools have some form of dynamic traffic assignment (DTA). Because of its demands on computer resources, DTA is often implemented at the mesoscopic level. DTA models are often embedded in traffic microsimulation tools.

DTA models are a special case of descriptive models that are based on an objective (minimize the travel time or disutility associated with a trip) that is gradually improved over a sequence of iterations until the network reaches a state of equilibrium.

The optimization process may be characterized as either *system-optimal* or *user-optimal.* A user-optimal solution does not necessarily produce an optimal result for the system as a whole and vice versa. With user-optimal models, the objective being applied reflects a behavioral assumption, and therefore the model is primarily descriptive. System-optimal models enforce some changes in driver behavior and are therefore normative. The formulation of the generalized cost (disutility) function can be expanded to reflect actual driving behavior more accurately—for example, by taking into account the number of stops or the driver's familiarity with typical traffic conditions.

The important point is that the analyst needs to know which type of model is being used and how that type influences the model's predictions. For example, assume that the analyst is dealing with a scenario in which the signal timing is fixed and drivers can alter their path choices in response to those signal timings (in a way that replicates how they would actually behave). This is a descriptive model and is a common application of a DTA model as mentioned above. Even though the analyst can change the signal timings and see how the drivers respond (and how the system performance changes), the model is still describing how the system would behave for a given set of conditions. On the other hand, if the analyst alters the scenario so that it seeks a better set of signal timings, a normative model has been created.

A descriptive model is implied if the analyst introduces a new demand–supply paradigm, such as congestion pricing, based on a field study. A new demand-side routine could be developed to predict how drivers alter path choices in response to congestion prices, and a supply-side routine could be developed that seeks to set those prices in some responsive and responsible way

in an effort to produce a desirable flow pattern. Even though two competing optimization schemes are at work, each describes how a portion of the system is behaving in response to inputs received. There is no explicit intent to optimize the system performance in a specific manner.

CONCEPTUAL DIFFERENCES BETWEEN DETERMINISTIC AND SIMULATION TOOLS

There are some conceptual differences between the HCM's analytical modeling and simulation modeling. It is useful to examine these differences before addressing alternative tool applications. Most of the differences may be described in terms of the way analytical and simulation tools deal with various traffic flow phenomena. Examples of the significant differences are identified in general terms in Exhibit 6-1.

APPROPRIATE USE OF ALTERNATIVE TOOLS

Situations when alternative tools might supplement HCM procedures.

Using alternative tools to supplement HCM capacity and quality-of-service procedures should be considered when one or more of these conditions apply:

- The configuration of the facility has elements that are beyond the scope of the HCM procedures. Each procedural chapter identifies the specific limitations of its own methodology.
- Viable alternatives being considered in the study require the application of an alternative tool to make a more informed decision.
- The measures produced by alternative tools are compatible with corresponding HCM measures and are arguably more credible than the HCM measures.
- The measures are compatible with corresponding HCM measures and are a by-product of another task, such as optimization of a network traffic control system.
- The measures are compatible with corresponding HCM measures and the decision process requires additional performance measures, such as fuel consumption and emissions, which are beyond the scope of the HCM.
- The system under study involves a group of different facilities or travel modes with mutual interactions invoking several procedural chapters of the HCM. Alternative tools are able to analyze these facilities as a single system.
- Routing is an essential part of the problem being addressed.
- The quantity of input or output data required presents an intractable problem for the HCM procedures.
- The HCM procedures predict oversaturated conditions that last throughout a substantial part of a peak period or queues that overflow the available storage space, or both.

Exhibit 6-1
Comparison of Methods for Addressing Traffic Phenomena by the HCM and Typical Microsimulation Tools

Traffic Phenomenon	Deterministic (HCM) Treatment	Typical Microsimulation Treatment
Right turn on red	Subtract right-turn-on-red volume from demand	Microscopic model of gap acceptance and follow-up time
Permitted left turns	Empirical model of capacity versus opposing volume, with minimum capacity determined by an assumption of two sneakers per cycle	Microscopic model of gap acceptance and follow-up time
STOP sign entry	Macroscopic model of gap acceptance and follow-up time	Microscopic model of gap acceptance and follow-up time
Channelized right turns	Subtract right-turning volume from demand	Microscopic model of gap acceptance and follow-up time; implicit effects of right-turn queues
Ramp merging	Empirical model of merge capacity versus freeway volume in the two outside lanes	Microscopic model of gap acceptance and follow-up time (some tools incorporate cooperative merging features)
Merging during congested conditions	Not addressed	Microscopic model of gap acceptance
Lane-changing behavior	Macroscopic model based on demand volumes and geometrics	Microscopic model of lane-changing behavior
Queue start-up on green	Fixed start-up lost time subtracted from the displayed green time	Stochastic lost time applied to the first few vehicles in the departing queue
Response to change interval	Fixed extension of green time added to the displayed green time	Kinematic model of stopping probability
Actuated signal operation	Deterministic model for computing green times as a function of demand and operating parameters	Embedded logic emulates traffic-actuated control explicitly; tools vary in the level of emulation detail
Delay accumulation	Analytical formulation for uniform delay based on the assumption of uniform arrivals over the cycle and uniform departures over the effective green	These three effects are combined implicitly in the accumulation and discharge of individual vehicles over the analysis period
Progression quality	Adjustment factor applied to the uniform delay term	
Random arrivals	Analytical formulation for incremental delay	
Generation of vehicles	Incremental delay formulation assumes Poisson arrivals (mean = variance) at the stop line; the variance–mean ratio is reduced for traffic-actuated control as a function of the unit extension	Individual vehicles are introduced into entry links randomly, on the basis of a specified distribution
Effect of oversaturation	A third analytical formulation, d_3, is introduced to cover the additional delay due to an initial queue	Oversaturated operation and residual queues are accounted for implicitly in the accumulation and discharge of individual vehicles
Residual queue at the end of analysis period	Analytical formulation computes the residual queue when d/c >1.0; the residual queue from one period becomes the initial queue for the next period	

In addition, when a specific HCM procedure has been developed by using simulation results as a surrogate for field data collection, it might be appropriate to use the underlying simulation tool directly to deal with complex configurations that are not covered in the HCM.

The following are factors to consider in the decision to use an alternative tool:

- Is the use of the alternative tool acceptable to the agency responsible for approving the decisions that result from its use?
- Are the necessary resources, time, and expertise available to apply the alternative tool?
- Does the application rely on a traceable and reproducible methodology?
- Have assumptions used to apply the tool been sufficiently documented?
- Is sufficient time available for calibration to promote a robust reliance on the model output?
- Are sufficient and appropriate data available to capitalize on or leverage the strength of the alternative tool?
- Are the alternative tool's performance measures (output) defined and computed in a manner consistent with the specification given in Chapter 7, Interpreting HCM and Alternative Tool Results?

Exhibit 6-2 provides examples of typical alternative tool applications for various situations that occur with both interrupted- and uninterrupted-flow conditions. Corridor and areawide analyses are also addressed in this exhibit. HCM procedures, which focus on points on the roadway and on linear roadway systems, tend to have limitations that are best addressed by tools that explicitly model corridors and areawide transportation systems.

For corridor and areawide analysis, aggregation of the results from Volume 2 and Volume 3 procedures require an intractable amount of data and effort. The interaction between individual facilities cannot be accounted for by HCM procedures. DTA might be required to balance flows among facilities and to model congestion propagation through the network due to oversaturation. Examples of promising alternative tool applications include active traffic management (ATM), congestion pricing, and freight corridors.

Alternative tool applications may be most appropriate for the evaluation of ATM measures that address operating conditions not explicitly addressed by HCM procedures. ATM consists of the dynamic and continuous monitoring and control of traffic operations on a facility. Examples of ATM measures include congestion pricing, managed lanes, ramp metering, changeable message signs, incident response, and speed harmonization (variable speed limits).

ATM measures, because of their dynamic nature, cannot be adequately modeled with HCM procedures. HCM procedures assume steady-state demand and control conditions within the typical minimum 15-min analysis period. Alternative tool applications that model traffic operations and interactions with traffic controls at the minute-by-minute level will be more accurate than HCM procedures that model operations at the higher level of temporal aggregation.

More information on ATM measures and their evaluation can be found in Chapter 35, Active Traffic Management.

Exhibit 6-2
Typical Applications for Alternative Traffic Analysis Tools

HCM Chapter	Typical Alternative Tool Application
Typical Applications in Volume 2: Uninterrupted Flow	
Applicable to all uninterrupted-flow procedures	Bottlenecks Oversaturated flow analysis Time-varying demands Unbalanced lane use Special lane restrictions Surveillance, work zones
10: Freeway Facilities	Managed lanes Surface street traffic control and ramp metering
11: Basic Freeway Segments	See uninterrupted-flow situations above
12: Freeway Weaving Segments	Complex weaving areas
13: Freeway Merge and Diverge Segments	Ramp metering High-occupancy-vehicle ramp lanes
14: Multilane Highways	See uninterrupted-flow situations above
15: Two-Lane Highways	Combination of terrain and traffic characteristics such as power–weight ratios or coefficient of variation of desired speeds
Typical Applications in Volume 3: Interrupted Flow	
Applicable to all interrupted-flow procedures	Oversaturated flow analysis (except for signalized intersections) Bus activity On-street parking Special lane use Queue spillback Pedestrian–bicycle interactions
16: Urban Street Facilities	Multimodal system analysis
17: Urban Street Segments	Mix of signals and no signals (stop and yield) Effects of midblock bottlenecks Signal timing plan development Turn bay overflow
18: Signalized Intersections	Geometrically offset intersections Alternative arrival characteristics Phase skips Pedestrian actuation Timing plan development
19: Two-Way STOP-Controlled Intersections	Two-way left turns Yield-controlled intersection delay TWSC intersection on a signalized arterial
20: All-Way STOP-Controlled Intersections	AWSC intersection on a signalized arterial
21: Roundabouts	Roundabout on a signalized arterial Multilane roundabouts Effect of geometrics Mixed-mode traffic
22: Interchange Ramp Terminals	Full cloverleaf interchange Backup from freeway segments Long-term (i.e., multicycle) blockage of approaches
23: Off-Street Pedestrian and Bicycle Facilities	Explicit modeling of pedestrian crossing activity

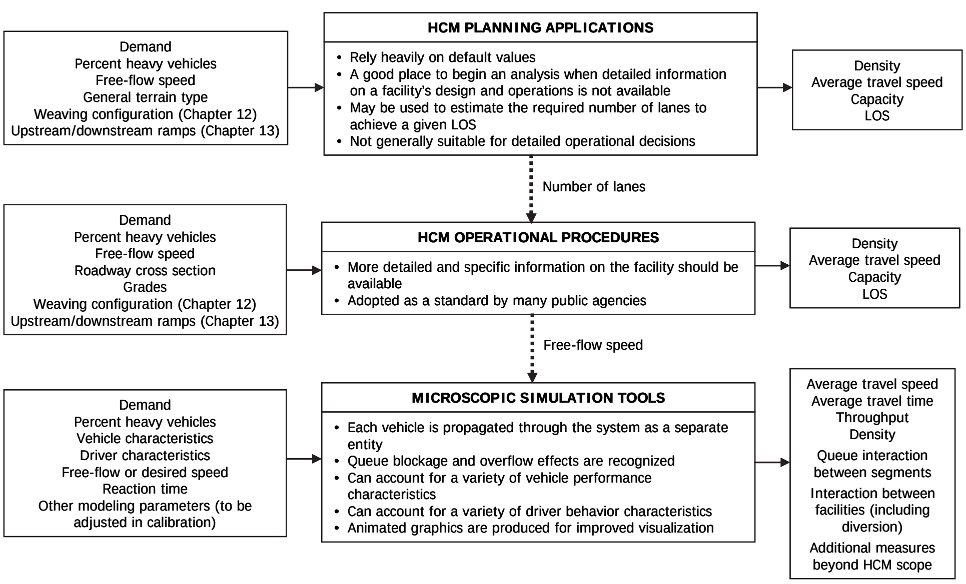
APPLICATION FRAMEWORK FOR ALTERNATIVE TOOLS

A wide range of tools is available for application to most highway capacity and performance analysis situations. Each tool has certain inputs and outputs, some of which can support a productive flow of information between tools. This section discusses the classes of tools that are available to address different types of system elements and suggests a framework for their application. Since the application framework differs among system elements, each element will be discussed separately. In developing input data for all these elements, it is important to reflect local default values.

Freeways

The modeling framework for freeways is presented in Exhibit 6-3. Each of the tools and procedures can be used in a stand-alone fashion; the potential flow of information between them indicates how they might fit into an overall analysis structure.

Exhibit 6-3
Freeway Modeling Framework for the HCM and Alternative Tools



Tools available for modeling freeways include HCM planning procedures, HCM operational procedures, and microscopic simulation.

Alternative tools find a much stronger application to freeway facilities than to individual freeway segments.

The principal classes of tools are

- The HCM planning applications, which rely heavily on default values for parameters;
- The HCM operational applications that require more detailed inputs in place of the default parameter values; and
- Microscopic simulation tools, as described previously in this chapter.

Most HCM freeway analysis limitations are apparent when the freeway is analyzed as a facility consisting of multiple segments of different types (basic segments, ramps, weaving segments, etc.) by using the procedures given in Chapter 10, Freeway Facilities. Alternative tools, especially microsimulation tools, find a much stronger application to freeway facilities than to individual segments.

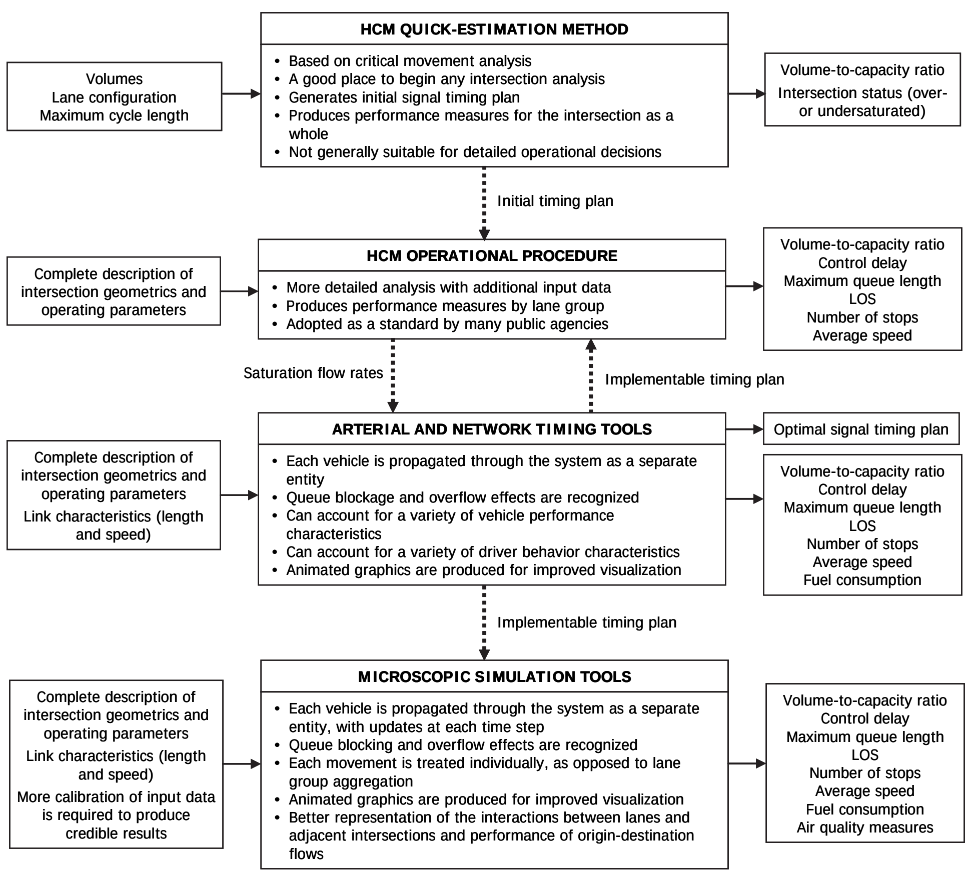
Urban Streets

Tools available for modeling urban streets include the HCM quick-estimation method for signalized intersections, HCM operational methods, arterial and network signal-timing tools, and microscopic simulation.

The modeling framework for urban streets, including their intersections, is presented in Exhibit 6-4. Each of the tools and procedures can be used in a stand-alone fashion; the potential flow of information between them indicates how they might fit into an overall analysis structure. The principal classes of tools are

- The HCM quick-estimation method for signalized intersections, which is based primarily on critical movement analysis and default values;
- The HCM operational methods for urban streets, including all types of intersections, which require more detailed traffic inputs and operating parameters;
- Arterial and network signal-timing tools, which produce recommended signal-timing plans based on measures that are generally similar to those produced by the HCM procedures; and
- Microscopic simulation tools, as described previously in this chapter.

Exhibit 6-4
Urban Street Modeling Framework for the HCM and Alternative Tools



Source: *Signalized Intersections: Informational Guide* (*11*).

Signal-timing tools generate signal-timing plans that can be used as inputs to HCM operational methods or to microsimulation tools.

Signal-timing tools are mostly based on macroscopic analytical models of traffic flow. Because they are the only class of urban street analysis tool that generates a signal-timing plan design, they are frequently used as an alternative tool for this purpose. The signal-timing plan may be fed into the HCM operational analysis or used as input to a microsimulation tool.

Microsimulation tools are used in urban street analysis, mainly to deal with complex intersection phenomena beyond the limitations of the HCM. These tools evaluate interactions between arterial segments, including the effect of various types of unsignalized intersections. They are also applied in evaluating networks and corridors with parallel facilities with the use of DTA routines.

Microsimulation tools are used to deal with complex intersection interactions beyond the limitations of the HCM.

Two-Lane and Multilane Highways

At this point, there is minimal application of alternative tools for the analysis of either type of highway. The HCM is the only macroscopic deterministic tool in common use, although some states such as Florida have developed their own analysis tools that implement derivatives of HCM procedures (*12*). At the time of writing, microsimulation models were in various stages of development. Some two-lane highway simulation tools were beginning to emerge, but there was insufficient experience to provide guidance for their use as an alternative to the methodology provided here.

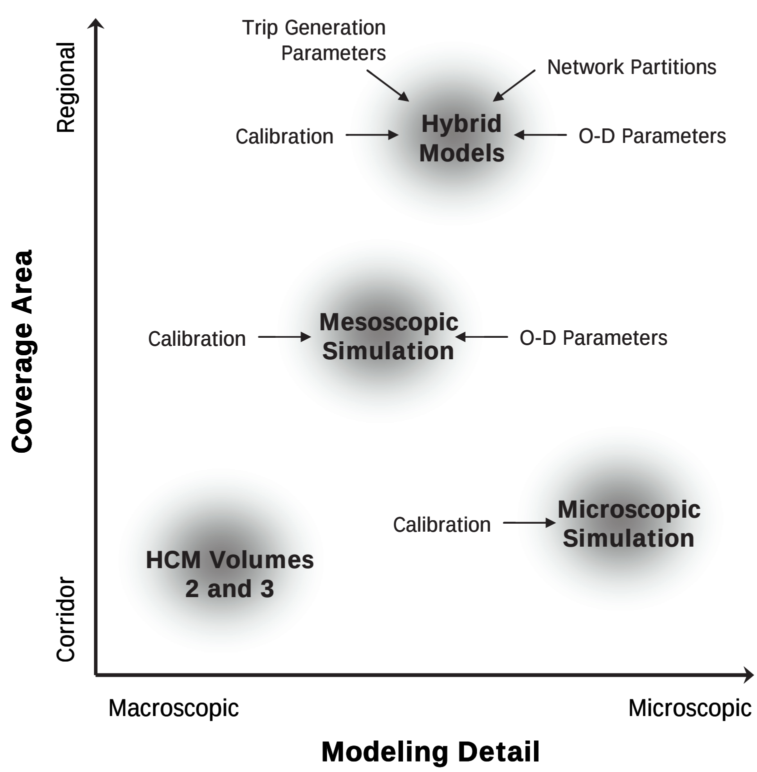
At the time of writing, the HCM was the only deterministic tool in common use for two-lane and multilane highways.

Corridor and Areawide Analysis

Corridor and areawide analysis is an important application for alternative tools. The HCM procedures deal mainly with points and segments and are limited in their ability to recognize the interaction between segments and facilities. The overall modeling framework for corridor and areawide analysis is presented in Exhibit 6-5, which shows the relationship of the HCM to the broad field of corridor and areawide analysis models.

Corridor and areawide analysis is probably the most important application for alternative tools.

Exhibit 6-5
Corridor and Areawide Analysis Modeling Framework for the HCM and Alternative Tools



An excellent reference for corridor and areawide simulation (*13*) is emerging from a U.S. Department of Transportation research initiative on integrated corridor modeling, providing detailed guidance on the conduct of large-scale simulation projects. This section presents an overview of corridor and areawide simulation from the perspective of HCM users, but considerably more detailed information is presented in the report (*13*), including a more detailed analysis framework.

The framework for corridor and areawide analysis differs from the framework presented for freeways and urban streets in three ways:

1. The HCM procedures account for a much smaller part of the modeling framework.
2. Different levels of simulation modeling are represented here. Simulation of urban streets and freeways is typically performed only at the microscopic level.
3. The framework is two-dimensional, with the coverage area as one dimension and the modeling detail as the other.

The selection of a model class (microscopic, mesoscopic, or hybrid) reflects a trade-off between coverage area and modeling detail.

The various model classes shown in Exhibit 6-5 depict the trade-off between these characteristics. The trade-off between coverage area and modeling detail is evident:

- Microscopic simulation provides more detail and more coverage than the HCM procedures. The additional detail comes from the microscopic nature of the model structure. The additional coverage comes from the ability to accommodate multiple links and nodes.
- Mesoscopic simulation provides more coverage with less modeling detail than microscopic simulation. In addition to accommodating larger areas, mesoscopic models are computationally faster than microscopic models and are thus well suited to the iterative simulations required for DTA, which can be time-consuming.
- Hybrid modeling uses network partitioning to treat more critical parts of the system microscopically and less critical parts mesoscopically—or even macroscopically. In this way, the regional coverage may be expanded without losing essential detail. A typical application for hybrid modeling might be interurban evacuation analysis, which must accommodate a very large geographical area without loss of detail at critical intersections and interchanges.

Some examples of corridor modeling are presented in Volume 4 of the HCM.

PERFORMANCE MEASURES FROM ALTERNATIVE TOOLS

The tool selected for a given analysis needs to provide performance measures that realistically reflect the attributes of the problem being studied.

Before the analyst can select the appropriate tool, the performance measures that realistically reflect attributes of the problem under study must be identified. For example, when oversaturated conditions are studied, it is necessary to use a tool that quantifies the effects of queuing as well as stops and delay. If the methodologies presented in Volumes 2 and 3 do not provide a particular performance measure of interest to the analyst (e.g., fuel consumption and emissions), an alternative tool might be required. Exhibit 6-6 provides a

summary of important performance measures for the procedures discussed in Volumes 2 and 3. The applicability of the HCM procedures and alternative tools is indicated for each chapter in this exhibit.

Exhibit 6-6
Principal Performance Measures from the HCM and Alternative Tools

Uninterrupted-Flow Chapters (Volume 2)								
HCM Chapter	**Speed**	**Delay**	**Through-put**	**Density**	**% Time Spent Following**	**Passing/ Overtaking**	**Environ-mental**	**Demand/ Capacity**
10. Freeway Facilities	H, A	H, A	H, A	H, A	X	X	H	X
11. Basic Freeway Segments	H, A	A	H, A	H, A	X	X	A	H
12. Freeway Weaving Segments	H, A	A	H, A	H, A	X	X	A	H
13. Freeway Merge and Diverge Segments	H, A	A	H, A	H, A	X	X	A	H
14. Multilane Highways	H, A	A	H, A	H, A	X	X	A	H
15. Two-Lane Highways	H, A	H	H	A	H	H, A	A	H
Interrupted-Flow Chapters (Volume 3)								
HCM Chapter	**Delay**	**Stops**	**Through-put**	**Queue Length**	**Cycle Failure**	**Environ-mental**	**Speed**	**Demand/ Capacity**
16. Urban Street Facilities	H, A	A	H, A	H, A	A	A	H, A	H
17. Urban Street Segments	H, A	A	H, A	H, A	A	A	H, A	H
18. Signalized Intersections	H, A	A	H, A	H, A	A	A	X	H
19. TWSC Intersections	H, A	A	H, A	H, A	X	A	A	H
20. AWSC Intersections	H, A	A	H, A	H, A	X	A	A	H
21. Roundabouts	H, A	A	H, A	H, A	X	A	A	H
22. Interchange Ramp Terminals	H, A	A	H, A	H, A	A	A	X	H
23. Off-Street Pedestrian and Bicycle Facilities	X	X	X	X	X	X	H	H (pedestrian)

Notes: Applicability codes:
- H: Performance measures computed by the HCM and some deterministic tools with similar computational structures.
- A: Performance measures computed by alternative tools (mostly simulation-based).
- X: Performance measures do not apply to this chapter.

TWSC: Two-way STOP-controlled. AWSC: All-way STOP-controlled.
Source: Adapted from Dowling et al. (*3*).

When an alternative tool is used to analyze highway capacity and quality of service, its performance measures should ideally be compatible with those prescribed by the HCM. Chapter 7 provides general guidance on this topic, while selected chapters in Volumes 2 and 3 provide specific guidance for certain system elements.

If an alternative tool is used to analyze highway capacity and quality of service, then the performance measures generated by the tool should, to the extent possible, be compatible with those prescribed by the HCM. Alternative tools frequently apply the same terminology to performance measures as the HCM, but divergent results are often obtained from different tools because of differences in definitions and computational methods. General guidance on reconciling performance measures is given in Chapter 7, Interpreting HCM and Alternative Tool Results. More specific guidance on dealing with performance measures from alternative tools is given in several of the procedural chapters in Volumes 2 and 3.

TRAFFIC ANALYSIS TOOL SELECTION CRITERIA

The success of a traffic analysis project depends on the selection of the best tool or tools for the purpose, followed by the proper application of the selected tools. Both of these issues are addressed in detail in the *Traffic Analysis Toolbox*, and the guidance provided in the *Toolbox* should be studied thoroughly before a major traffic analysis project is undertaken.

Determining Project Scope

Questions to ask during the scoping of a traffic analysis project.

A properly defined problem and project scope are essential prerequisites to the correct selection of tools or procedures for the project. Answers to the following questions will assist in scoping the project:

1. What is the operational performance problem or goal of the study?
2. Does the network being studied include urban streets, freeways, rural highways, or any combination of them?
3. Are multiple routes available to drivers?
4. What are the size and topology (isolated junctions, linear arterial, grid) of the network?
5. What types of roadway users (cars, carpools, public transit vehicles, trucks, bicycles, pedestrians) should be considered?
6. What traffic control methods (regulatory signs, pretimed signals, actuated signals, real-time traffic-adaptive signals, and ramp-metering signals) should be considered?
7. Should oversaturated traffic conditions be considered?
8. Does the network involve specialized traffic control or intelligent transportation system (ITS) features that are not covered by the HCM?
9. What is the duration of the analysis period?
10. Do the geometric conditions of the roadway facility change during the analysis period?
11. Does the traffic demand fluctuate significantly during the analysis period?
12. Does the traffic control change during the analysis period?
13. What output and level of detail are anticipated from the tool?
14. What information is available for model input, model calibration, and validation?
15. Are multiple methods available for consideration in the analysis?

Assessing HCM Methodologies

Use the Limitations of the Methodology and Special Cases sections of Volume 2 and 3 chapters to assess the appropriateness of the HCM methodology for a given analysis.

Another essential step in the analysis tool selection process is to assess the capability of the existing HCM methodologies and to determine whether these methodologies can be applied (in whole or in part) to the issues that were raised in the project-scoping step. In addressing these issues, two major questions should be answered: What are the limitations of the HCM methodologies? Can the limitations be overcome? Limitations of the existing HCM methodologies for each facility type are identified in the procedural chapters of Volumes 2 and 3 of this manual. If it is determined that an alternative tool is needed or advisable, then the most appropriate tool must be selected.

Selecting a Traffic Analysis Tool

Every traffic analysis tool, depending on the application, has its own strengths and weaknesses.

Each analytical or simulation model, depending on the application, has its own strengths and weaknesses. It is important to relate relevant model features to the needs of the analysis and determine which tool satisfies those needs to the greatest extent. Both deterministic and simulation-based tools could be candidates for overcoming HCM limitations. In most cases, however, deterministic tools will exhibit limitations similar to the HCM procedures, which

are also deterministic. Deterministic tools also tend to work at the same macroscopic level as the HCM. The main applications for alternative deterministic tools fall into the following categories:

- Signal-timing plan design and optimization,
- Proprietary deterministic models that offer features not found in the HCM,
- Proprietary tools that exchange data with other traffic analysis software, and
- Roundabout analysis tools that deal with geometric and operational parameters beyond the scope of the HCM.

Simulation tools will generally be chosen to deal with situations that are too complex to model as a deterministic process. The balance of this discussion deals primarily with microscopic simulation tools.

Model Development Environment and Process

The manner in which the modeling logic was developed is an important consideration in the selection of a traffic analysis tool. The credibility of results from a simulation model depends on the process by which it was designed, implemented, and tested. A comprehensive document (*14*) describes "a formal process used to verify the model's reproducibility of traffic conditions in the development of a traffic flow simulation model." The intent is to prescribe a set of requirements to which all tools should conform. There is no attempt to compare the merits of different tools.

The proper simulation model development process is described in terms of the following steps. The desirable attributes of each of these steps are described:

- Determination of model specifications,
- Contrivance of the model operation principles,
- Programming and debugging,
- Verification using virtual data, and
- Verification using actual data.

The traffic phenomena to be modeled through simulation are identified, and the theoretical basis for dealing with each one in a traffic model is described:

- Generation of vehicles,
- Bottleneck capacity or saturation flow rate at a link's downstream end,
- Drawing and elimination of breakdown and shock wave propagation speed,
- Capacity of the merge/diverge area and the merge/diverge ratio,
- Decrease in the left-turn capacity due to opposing traffic in a signalized intersection, and
- Route selection behavior.

Users who are unfamiliar with the operation of their candidate simulation tools may wish to consult this document for informational purposes.

Key model features to consider.

Model Capabilities

A review of modeling capabilities is probably the most important aspect of selecting a simulation tool. Simulation model results may be of no value if the model is not capable of addressing the project-scoping issues raised in the initial step. Some key features can be used to evaluate a model's capabilities, such as size of network, network representation, traffic representation, traffic composition, traffic operations, traffic control, and model output.

- *Size of network.* Most tools limit the size of network that can be analyzed and the number of vehicles that can be accommodated. The key network parameters limited by the tool include number of nodes, number of links, number of lanes per link, and number of sign- or signal-controlled intersections.
- *Network representation.* Network representation refers to the tool's ability to represent the network geometries for urban streets, freeways, rural highways, or any combination, ranging from single intersections to grid networks. For urban streets, major geometric elements include lane channelization at intersections, turning pockets, and bus bays and stops. For freeways, major geometric elements are acceleration lanes, deceleration lanes, auxiliary lanes, on-ramps, off-ramps, lane additions, lane drops, horizontal curvature, and grade. Elements for rural highways include grade, curvature, passing and no-passing zones, and sight distance for overtaking and passing.
- *Traffic representation.* The representation of how traffic flows in the model, especially the level of aggregation, is an important consideration. Because of their microscopic and stochastic nature, microscopic models can simulate sophisticated vehicle movements, allowing analysts to perform complex traffic analyses such as those for weaving areas. In contrast, mesoscopic and macroscopic models are generally not appropriate for evaluating complex traffic conditions, since they use aggregate measures of flow or density to describe vehicle movements.
- *Traffic composition.* Traffic composition represents the mix of cars, buses, trucks, carpools, and bicycles in the network and is used to incorporate the differences in performance characteristics among types of vehicles and modes. Special attention needs to be given to the selection of the analysis tool when networks with dedicated accommodation for nonautomobile modes are involved.
- *Traffic operations.* The model should be capable of representing real-world traffic operations such as complex merging, diverging, and weaving maneuvers at interchanges, high-occupancy-vehicle lanes, bus transit operations, lane channelization at intersections, lane restrictions, lane blockages, and parking activities.
- *Traffic control.* For street intersections, control methods include YIELD signs, two-way STOP signs, all-way STOP signs, pretimed signals, actuated signals, real-time traffic-adaptive control signals, and traffic-responsive control systems. Those commonly used for freeway on-ramps include pretimed control, demand and capacity control, occupancy control, speed

control, high-occupancy-vehicle priority at ramps, integrated (areawide) ramp control, ramp-metering optimization, and dynamic real-time ramp-metering control with flow prediction capabilities. Signal coordination between traffic signals or between on-ramp signals and traffic signals on adjacent streets may also need to be considered.

- *DTA features.* When a network involves multiple routes that present a choice to the driver, the model must employ dynamic assignment logic to distribute the vehicles over the available paths in a realistic way. Simulation models offer varying degrees of DTA features and may allow for various influencing factors to be included in the decision process to reflect driver behavior. DTA models are discussed in more detail later in this chapter.
- *Other ITS devices.* In addition to the ITS elements in the traffic control category, tools may be able to model the effects of other ITS devices, such as in-vehicle navigation systems, dynamic message signs, incident management, smart work zones, or intervehicle communication technologies.
- *Real-time process control features.* Many tools offer the ability to communicate directly with other processes invoked in either hardware or software. Examples include intersection signal controllers and large-scale network traffic management systems. Most highway capacity analysis projects will not require features of this type. However, when complex networks with ITS elements are involved, the ability of a simulation tool to communicate directly with the outside world might become a significant factor in the selection of the proper tool.

Above all, the analyst should review the user's guide for the selected tool to get a more detailed description of its characteristics.

User Interface

User interface considerations.

The user interface includes all of the features of a tool that supply input data from the user to the model and output data from the model to the user. Simulation tools vary in the nature of their user interface. To some extent, the suitability of the user interface is a matter of individual preference. However, a highly developed user interface can offer a better level of productivity for larger and more repetitive tasks. Selection criteria related to the user interface include

- The amount of training needed to master its operation,
- The extent to which it contributes to productive model runs,
- The extent to which it is able to import and export data between other processes and databases, and
- Special computational features that promote improved productivity.

The following are the principal elements associated with the user interface:

- *Inputs.* Most of the inputs required by the model will be in the form of data. In most cases, the input data will be entered manually. Most tools offer some level of graphic user interface to facilitate data entry. Some tools also offer features that import data directly from other sources.

- *Outputs.* Two types of outputs are available from simulation tools: graphics files and static performance measures. Graphics files provide graphics output, including animation, so that users can visually examine the simulation model results. Static performance measures provide output for numerical analysis. Both types of outputs may be presented directly to the user or stored in files or databases for postprocessing by other programs.
- *Multiple-run support*. The stochastic nature of simulation models requires multiple runs to obtain representative values of the performance measures. Chapter 7, Interpreting HCM and Alternative Tool Results, provides guidance on the number of runs required under specific conditions. The ability of a tool to support multiple runs is an important selection criterion. Multiple-run support includes processing functions that perform a specified number of runs automatically and postprocessing functions that accumulate the results from individual runs to provide average values and confidence intervals.

Data Availability

Consider the kind of data required and the availability of the data in selecting a tool.

The next criterion identifies data requirements and potential data sources so that the disparity between data needs and data availability can be identified. In general, microscopic models require more intensive and more detailed data than do mesoscopic and macroscopic models. Three different types of data are required to make the application of the traffic simulation model successful: data for model input, data for model calibration, and data for model validation.

Data for Model Input

The basic data items required to describe the network and the traffic conditions to be studied can be categorized into four major groups:

1. *Transportation network data.* Simulation tools incorporate their network representation into the user interface, and some differences are observed among different tools. Most simulation models use a link-based scheme in which links represent roadway segments that are connected in some manner. The required link data include endpoint coordinates, link length, number of lanes per link, lane additions, lane drops, turning pockets, lane channelization at intersections, grade, and horizontal curvature. The connector data describe the manner in which the links are connected, including the permissible traffic movements, type of control, and lane alignment.
2. *Traffic control and ITS data.* Detailed control data should be provided for all control points, such as street intersections or freeway on- and off-ramps. Sign controls include YIELD signs, two-way STOP signs, and all-way STOP signs. Signal controls include pretimed signals, actuated signals, or real-time traffic-adaptive signals. Ramp-metering control methods include all of the modes described earlier. Timing data are required for all signal controls. Detector data such as type and location of the detector are required for actuated and traffic-adaptive signals. Any special ITS

features involved in a project will create a need for additional data describing their parameters.

3. *Traffic operations data.* To represent the real-world traffic environment, most simulation tools take link-specific operations data as input, such as parameters that determine roadway capacity, lane use, lane restriction, desired free-flow speed, high-occupancy-vehicle lanes, parking activities, lane blockages, and bus transit operations.
4. *Traffic demand data.* Different tools may require traffic demand data in different formats. The most commonly used demand data are traffic demand at the network boundary or within the network, traffic turning percentages at intersections or freeway junctions, O-D trip tables, path-based trips between origins and destinations, and traffic composition.

Data for Model Calibration

Calibration adjusts a model's vehicle, driver, and other characteristics so that the model can realistically represent the traffic environment being analyzed.

Calibration was defined previously as the process by which the analyst selects the model parameters that cause the model to best reproduce field-measured local traffic operations conditions. Vehicle and driver characteristics, which may be site-specific and require calibration, are the key parameters. These data take the form of scalar elements and statistical distributions that are referenced by the model. In general, simulation models are developed and calibrated on the basis of limited site-specific data. The development data may not be transferable and therefore may not accurately represent the local situation. In that case, the model results should be interpreted with caution, and the default parameters that must be overridden to better reflect local conditions should be identified. Most simulation tools allow the analyst to override the default driver behavior data and vehicle data to better match local conditions, thereby allowing for model calibration. The calibration process should be documented, traceable, and reproducible to promote a robust analysis.

1. *Driver behavior data.* Driver behavior is not homogeneous, and thus different drivers may behave differently in the same traffic conditions. Most microscopic models represent stochastic or random driver behavior (from passive to aggressive drivers) by taking statistical distributions of behavior-related parameters such as desired free-flow speed, queue-discharge headway, modeling parameters for lane changing and car following, and driver response to advance information and warning signs.
2. *Vehicle data.* Vehicle data represent the characteristics and performance of the types of vehicles in the network. Different vehicle types (e.g., cars, buses, single-unit trucks, semitrailers) have different characteristics and performance attributes. They vary in terms of vehicle length, maximum acceleration, maximum deceleration, emissions rate, and fuel consumption rate. All traffic simulation tools provide default vehicle characteristics and performance data. These data need to be overridden only when the local vehicle data are known to be different from the default data provided by the tool or when the default values do not provide reasonable results.

Data Sources

Data collection is costly. Analysts should explore all possibilities to leverage previously collected data. The analyst should identify which data are currently available and which data need to be collected in the field. Most static network, traffic, and control data can be collected from local agencies. Such data include design drawings for geometries, signal-timing plans, actuated controller settings, traffic volume and patterns, traffic composition, and transit schedules.

Ease of Use

Simulation models use assumptions and complex theories to represent the real-world dynamic traffic environment. Therefore, an input–output graphical display and debugging tools that are easily understandable are important criteria to consider in selecting a tool. Although ease of use is important in a simulation tool, the fact that a particular tool is easy to use does not necessarily imply that it is the correct choice. The following five criteria can be considered in assessing the ease of use of a simulation tool:

- *Preprocessor:* input data handling (user-friendly preprocessor);
- *Postprocessor:* output file generation for subsequent analysis;
- *Graphics displays:* graphic output capabilities, both animated and static;
- *Online help:* quality of online help support; and
- *Calibration and validation:* ability to provide guidelines and data sets for calibration and validation.

Required Resources

The following issues with regard to resources should be addressed in selecting a traffic analysis tool:

- *Costs to run the tool.* Examples are costs for data collection and input preparation, hardware and software acquisition, and model use and maintenance.
- *Staff expertise.* Intelligent use of the tool is the key to success. The analyst should understand the theory behind the model to eliminate improper use and avoid unnecessary questions or problems during the course of the project.
- *Technical support.* Quality and timely support are important in the acquisition of a tool.

User Applications and Past Performance

Credibility and user acceptance of a tool are built on the tool's past applications and experiences. No tool is error-free when first released, and all require continuous maintenance as well as periodic enhancements.

Verification and Validation

It is important to assess how extensively a given model has been verified. In many cases, but not all, simulation models are also validated as part of the formulation and development process. Certainly those that have been validated

are nominally better to use than those that have not been validated. Generally, models that have been in use for some time are likely to have been assessed and validated by various researchers or practitioners who are using them. It is useful to look for evidence of validation in professional journals and periodicals.

APPLICATION GUIDELINES FOR SIMULATION TOOLS

This section presents general guidance for the use of simulation-based traffic analysis tools for capacity and performance analysis. More detailed guidance for the application of these tools to specific facilities is presented in the procedural chapters of Volumes 2 and 3. Additional information, including sample applications, may be found in Volume 4 and in the *Traffic Analysis Toolbox* (*3*) mentioned earlier in this chapter.

After the project scope has been determined and the tool has been selected, several steps are involved in applying the tool to produce useful results.

Assembling Data

Data assembly involves collecting the data required (but not already available) for the selected tool. Data collection is costly. Analysts should capitalize on previous modeling efforts and identify data available through local agencies. When existing data are assembled, users should develop a comprehensive plan for collecting data that are missing. In some cases, a pilot data collection effort may be needed to ensure that the developed data collection plan is workable before a full-scale effort is conducted.

An important part of the data assembly process is a critical review of all data items to ensure the integrity of the input data set. Of special concern are the continuity of traffic volumes from segment to segment and the distribution of turning movements at intersections and ramp junctions. Each data item should be checked to ensure that its value lies within reasonable bounds.

Entering Data

Once all required data are in hand, the next step is to create the input files in a format required by the selected tool. The following are the most commonly used methods for creating input files:

- *Importing from a traffic database.* Many analysts have large amounts of data in a variety of formats for the general purpose of traffic analysis. Such databases can be used to create input files.
- *Converting from the existing data of other tools.* Many traffic models use the same or similar data for modeling purposes so that these data may be shared. Some traffic simulation tools are accompanied by utility programs that allow the user to convert data into input files required by other tools.
- *Entering the data from scratch.* Many traffic analysis tools have their own specific input data preprocessors, which aid the analyst with input data entry and review. These advanced features of the input data preprocessor eliminate cumbersome coding efforts. In addition, some input preprocessors include online help features.

Calibrating and Validating Models

The model should be run with the data set describing the existing network and traffic scenarios (i.e., the baseline case), and the simulation results should then be compared with the observed data collected in the field. The primary objective of this activity is to adjust the parameters in the model so that simulation results correspond to real-world situations.

Several critical issues must be addressed when an initial simulation model run is conducted for the baseline case. First, the model should be able to represent the initial state of the traffic environment before any statistics can be collected for analysis. Second, the time should be long enough to cover the entire analysis period. Third, if the model can handle time-varying input, the analyst should specify, to the extent possible, the dynamic input conditions that describe the traffic environment. For example, if 1 h of traffic is to be simulated, the analyst should always specify the variation in demand volumes over that hour at an appropriate level of detail rather than specifying average, constant values of volume.

Finally, the analyst should know how to interpret the simulation model results, draw inferences from them, and determine whether they constitute a reasonable and valid representation of the traffic environment. Given the complex processes taking place in the real-world traffic environment, the user must be alert to the possibilities that the model's features may be deficient in adequately representing some important process; that the specified input data, calibration, or both are inaccurate or inadequate; that the results provided are of insufficient detail to meet the project objectives; that the statistical analysis of the results is flawed (as discussed in the following section); or that the model has bugs or that some of its algorithms are incorrect, thereby necessitating revision. If animation displays are provided by the model, this option should always be exercised to identify any anomalies.

If the simulation model results do not reasonably match the observed data collected in the field, the user should identify the cause-and-effect relationships between the observed and simulated data and the calibration parameters and perform calibration and validation of the model. Information on calibration and validation of models may be found in the *Traffic Analysis Toolbox* (*1*–*3*) and in the references listed in Volume 4.

Special Considerations for DTA

The term "traffic assignment" traditionally refers to the process of computing path demands, or path input flows, given a network and an O-D demand matrix (trip table). In microscopic simulation models, this process is implemented as a route-choice model that is executed independently for each driver (vehicle) in the simulation. Using explicit routes to move vehicles through the simulation obviates the use of turning proportions at nodes. Routes and route flows may also be implicitly represented in a model by splitting rates, which are turning proportions at nodes by destination. Route flows can have a significant impact on model outputs such as LOS, since they play a key role in determining the local traffic demand on any given section of road.

Regardless of the implementation, traffic assignment is relevant whenever demand is defined in the form of an O-D matrix (static or time-varying) and multiple routes are available for some O-D pairs. It is particularly relevant when congestion affects the travel times on some of these routes. DTA produces time-varying path flows (or splitting rates) by using a dynamic traffic model that is either mesoscopic or microscopic. DTA models normally permit the demand matrix to be time-varying as well. The assignment model (routing decision) is based on a specific objective, which is predominantly the minimization of travel time, but will also take other factors such as travel cost (e.g., tolls or congestion pricing) and travel distance into consideration.

A fundamental issue concerning the role of travel time in route choice is that the actual travel time from origin to destination cannot be known in advance: it results from the collective route choices of all the drivers. Thus, the input to the routing decision (travel time) depends on the decision itself (route choice), forming a logical cycle. This type of problem can be solved with an iterative algorithm that repeats the simulation several (or many) times over, imitating the day-to-day learning process of drivers in the real world. At each iteration (or "day") the assignment is adjusted until the route-choice decisions are consistent with the experienced travel times: this is referred to as the user-optimal solution. In practice, an approximation to the user-optimal route flows is often determined by using a "one-pass" (noniterative) assignment in which drivers repeatedly reevaluate their routes during a single simulation run. The choice of method depends on network characteristics and modeling judgment.

The assignment (routing) component of a DTA model may be deterministic or stochastic in nature, independent of whether the traffic model is deterministic or stochastic. In general, both approaches can produce good results as long as they produce route choices that are consistent with the routing objective, for example, the minimization of generalized travel cost. The generalized cost is determined from the combination of a range of factors, such as travel time, travel distance, and direct costs (e.g., tolls), by applying relative weights to each of these factors, which typically differ by user class.

DTA applications are not trivial. Whereas single route applications are typically implemented by one analyst, DTA applications to large-scale systems are more likely to involve a team of analysts with a broader range of skills and experience. A comprehensive reference on DTA has been written by the Transportation Research Board's Committee on Network Modeling (*15*).

Analyzing Output

Proper output analysis is one of the most important aspects of any study using a simulation model. A variety of techniques are used, particularly for stochastic models, to arrive at inferences that are supportable by the output.

When the model is calibrated and validated, the user can conduct a statistical analysis of the simulation model results for the baseline case with calibrated parameters. If the selected simulation model is stochastic in nature, simulation model results produced by a single run of the model represent only point estimates in the sample population. Typical goals of data analysis using output

from stochastic-model experiments are to present point estimates of the performance measures and to form confidence intervals around these estimates. Point estimates and confidence intervals for the performance measures can be obtained from a set of replications of the system by using independent random number streams. The analyst should refer to previous studies for details on the design and analysis of stochastic simulation models in the *Traffic Analysis Toolbox* (*1*–*3*) and in Volume 4 of the HCM.

Analyzing Alternatives

When satisfactory simulation model results are obtained from the baseline case, the user can prepare data sets for alternative cases by varying geometry, controls, and traffic demand. If the model is calibrated and validated on the basis of the observed data, values of the calibrated parameters should also be used in the alternatives analysis, assuming that driver behavior and vehicle characteristics in the baseline case are the same as those in the alternative cases.

Traffic simulation models produce a variety of performance measures for alternatives analysis. As discussed previously, the user should identify what model performance measures and level of detail are anticipated. These performance measures, such as travel time, delay, speed, and throughput, should be quantifiable for alternatives analysis. Some tools provide utility programs or postprocessors, which allow users to perform the analysis easily. If animation is provided by the tool, the user can gain insight into how each alternative performs and can conduct a side-by-side comparison graphically.

4. REFERENCES

Many of these references can be found in the Technical Reference Library in Volume 4.

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APPENDIX A: DEVELOPING LOCAL DEFAULT VALUES

A default value is a constant to be used in an equation as a substitute for a field-measured (or estimated) value. Default values can be used for input parameters or calibration factors. The value selected should represent a typical value for the conditions being analyzed. Default values are generally used for planning, preliminary engineering, or other applications of the HCM that do not require the accuracy provided by a detailed operational evaluation (*A-1*). They can be applied to any of the modes addressed by the HCM.

Default values are generally used for HCM applications that do not require the accuracy provided by a detailed operational evaluation.

Local default values can be developed by conducting measurements of "raw data" in the geographic area where the values are to be applied. Typically, default values are developed for roadway or traffic characteristics to identify typical conditions of input variables for planning or preliminary engineering analysis. Default values should not be applied for input variables that can significantly influence the analysis results. For interrupted-flow facilities, these sensitive input variables include peak hour factor, traffic signal density, and percent heavy vehicles. For uninterrupted-flow facilities, these sensitive input parameters include free-flow speed, peak hour factor, and specific grade. In developing generalized service volume tables for daily service volumes, the *K*- and *D*-factors selected must be consistent with measured local values.

Default values should not be applied for input variables that can significantly influence the analysis results.

When local default values are developed, the raw data should be collected during the same time periods that will be used for analysis—typically during weekday peak periods. In some cases, the peak 15-min period is recommended as the basis for computation of default values because this time period is most commonly used for capacity and LOS analysis.

Input parameters that describe the facility type, area type, terrain type, and geometric configuration (such as lane width, segment length, and interchange spacing) are readily available to the analyst. Default values for these parameters should not be used.

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This reference can be found in the Technical Reference Library in Volume 4.

APPENDIX B: DEVELOPING LOCAL SERVICE VOLUME TABLES

INTRODUCTION

This appendix focuses on the automobile mode. To the limited extent that modal demand is an input to nonautomobile modes' LOS procedures, this material could also be applied to nonautomobile modes.

As discussed in the body of this chapter, service volume tables can provide an analyst with an estimate of the maximum number of vehicles a system element can carry at a given LOS. The use of a service volume table is most appropriate in certain planning applications in which evaluation of every segment or node within a study area is not feasible. Once potential problem areas have been identified, other HCM tools can be used to perform more detailed analyses for just those locations of interest.

To develop a service volume table, the analyst needs to develop a default value for each of the input parameters. The choice of default value can have a significant impact on the resulting service volume for a given LOS. For this reason, great care should be used to develop default values that the analyst believes are most appropriate for the local condition. When results are particularly sensitive to a particular input parameter, a range of default values should be considered for that parameter. The application of sensitivity analyses is discussed in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results.

A specific roadway's characteristics are unlikely to match exactly the default values used to generate a service volume table. Therefore, conclusions drawn from such tables should be considered to be rough approximations.

When the service volume table is applied, it is important to recognize that not all of the input parameters for the various roadway segments being evaluated are likely to match the default inputs. Accordingly, conclusions drawn from the application of service volume tables should be considered to be, and presented as, rough approximations.

TABLE CONSTRUCTION PROCESS

Service volume tables can be generated for a given system element by selecting appropriate input parameter values and applying software to back-solve for the maximum volume associated with a particular LOS. The procedure is as follows (*B-1*):

1. Determine all of the nonvolume default values to be used in developing the service volume table (e.g., number of lanes, peak hour factor, percentage of heavy vehicles, area type, *K*- and *D*-factors), in accordance with the guidance in Appendix A. Repeat the following steps for each system element and area category.
2. Identify the threshold value associated with the system element's MOE for LOS A by using the LOS exhibit in the Volume 2 or Volume 3 chapter that covers that system element. For example, a density of 11 pc/mi/ln is the maximum density for LOS A for a basic freeway segment.
3. Compute the MOE for a volume of 10 veh/h for an hourly volume table, or 100 veh/h for a daily volume table. If the result exceeds the LOS A threshold value, then LOS A is unachievable. Repeat Steps 2 and 3 for the next LOS (e.g., LOS B) until an achievable LOS is found, then continue with Step 4.

4. Adjust the input volume until the highest volume that achieves the LOS is found. Test volumes should be a multiple of at least 10 for hourly volume tables and a multiple of at least 100 for daily volume tables. If the table is being constructed by manually applying software, the analyst can observe how closely the MOE result is converging toward the LOS threshold value and can select a test volume for the next iteration accordingly. If the table generation function is being added to software, the automated method described below can be used to converge on the service volume progressively.
5. Identify the threshold value for the next LOS and repeat Steps 4 and 5 until threshold volumes (or unachievability) have been determined for each LOS.
6. If a daily volume table is being created, divide the identified hourly threshold volumes by the selected *K*- and *D*-factors and round down to a multiple of at least 100.
7. If desired, change the input value used for one of the input parameters (e.g., number of lanes) and repeat Steps 2 through 5 as many times as needed to develop service volumes for all desired combinations of input values.

The following is an automated method for finding threshold values:

1. Let the first achievable test volume be labeled Vol 1.
2. Select a second iteration volume (label this Vol 2) by doubling Vol 1.
3. Compute the MOE value for Vol 2.
4. If the resulting MOE value is lower than the LOS threshold, replace Vol 1 with Vol 2 and select a new Vol 2 with double the current Vol 2 value. Repeat Steps 3 and 4 until the MOE result is greater than the desired LOS threshold.
5. Use the bisection method described below (*B-1*) or another more efficient numerical method to converge on the service volume.
6. Compute the volume halfway between Vol 1 and Vol 2. Label this volume Vol 3.
7. Compute the MOE value for Vol 3.
8. If the MOE result for Vol 3 is greater than the desired LOS threshold, replace Vol 2 with Vol 3.
9. If the LOS result for Vol 3 is lower than the desired LOS threshold, replace Vol 1 with Vol 3.
10. Is the range between Vol 1 and Vol 2 acceptable? If yes, stop and use the average of Vol 1 and Vol 2. If not, repeat Steps 6 through 9.
11. If an hourly volume table is being generated, round down the result of Step 10 to a multiple of at least 10. If a daily volume table is being generated, divide the result of Step 10 by the selected *K*- and *D*-factors and round the result down to a multiple of at least 100.

This reference can be found in the Technical Reference Library in Volume 4.

REFERENCE

B-1. Courage, K. G., and J. Z.-Y. Luh. Computation of Signalized Intersection Service Volumes Using the 1985 Highway Capacity Manual. In *Transportation Research Record 1194*, Transportation Research Board, National Research Council, Washington, D.C., 1988, pp. 179–190.

CHAPTER 7
INTERPRETING HCM AND ALTERNATIVE TOOL RESULTS

CONTENTS

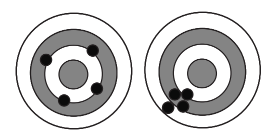
LIST OF EXHIBITS

1. INTRODUCTION

The evolution of computer software, with an ever-increasing variety of tools available, makes it easy for analysts to conduct transportation analyses and to consider a wide variety of factors in doing so. One thing that has not changed, however, is the analyst's responsibility to have a full understanding of the methodologies used by the selected analysis tools—including the level of uncertainty in the tools' results—in order to make well-informed recommendations based on the analysis results and to communicate those results to others. As tools become more complex and sophisticated, the analyst's challenge increases.

Uncertainty, variability, accuracy, and precision are related concepts that need to be considered when interpreting and presenting model results.

Chapter 7, Interpreting HCM and Alternative Tool Results, begins with a discussion of the *uncertainty* in model outputs that results from (*a*) uncertainty in a model's inputs, (*b*) uncertainty in the performance measure estimate produced by a model, and (*c*) imperfect model specification, in which a model may not fully account for all the factors that influence its output. Uncertainty in model inputs can result from the *variability* of field-measured values, from the uncertainty inherent in forecasts of future volumes, and from the use of default values. The *accuracy* of a model's results is directly related to its uncertainty. A model that incorporates more factors may appear on the surface to be more accurate, but if the inputs relating to the added factors are highly uncertain, accuracy may actually be decreased. Analysts should also carefully consider the *precision* used in presenting model results to avoid implying more accuracy than is warranted.



Alternative tools often provide performance measures that have the same, or similar, names as HCM measures but that are defined differently.

When both HCM-based and alternative tools are used in an analysis, or when a performance measure produced by an alternative tool is being used to determine level of service (LOS), analysts should take care to ensure that the measures produced by the alternative tool are defined in the same way as the HCM measure. Different tools use different definitions for similarly named measures, which may lead to inaccurate conclusions if the differences are not accounted for properly. Chapter 7 provides guidance on defining, measuring, and comparing key outputs of alternative tools when such outputs are intended to be used with or compared with those of the HCM.

Chapter 7 concludes with guidance on the number of significant digits to use in displaying HCM results and on ways to present HCM results to facilitate their interpretation by technicians, decision makers, and the general public.

2. UNCERTAINTY AND VARIABILITY

UNCERTAINTY AND VARIABILITY CONCEPTS

Model outputs—whether from the HCM or alternative tools—are estimates of the "true" values that would be observed in the field. Actual values will lie within some range of the estimated value.

The performance measure results produced by traffic models—both HCM based and alternative tools—are estimates of the "true" values that would be observed in the field. These estimates are not exact, however—they are subject to statistical *uncertainty,* in which the true value of a given measure lies within some range of the estimated value.

To illustrate the lack of exactness, consider the *variability* in measured values, such as traffic volume inputs. There are several types of variability:

Sources of variability in correctly measured values used as model inputs. Measurement error is yet another form of uncertainty.

- *Temporal variability*, in which measured values, such as hourly traffic volumes, vary from day to day or month to month at a given location;
- *Spatial variability*, in which measured values, such as the percentage of trucks in the traffic stream, vary from one location to another within a state or vary from one state to another; and
- *User perception variability*, in which different users experiencing identical conditions may perceive those conditions differently—for example, when asked to rate their satisfaction with those conditions.

Sources of uncertainty in model outputs.

Chapter 5, Quality and Level-of-Service Concepts, noted that model outputs are subject to three main sources of uncertainty (*1*):

1. Uncertainty in model inputs, such as variability in measured values, measurement error, uncertainty inherent in future volume forecasts, and uncertainty arising from the use of default values;
2. The uncertainty of the performance measure estimate produced by a model, which in turn may rely on the output of another model that has its own uncertainty; and
3. Imperfect model specification—a model may not fully account for all the factors that influence the model output.

Uncertainty cannot be eliminated, but its effects can be reduced through a variety of techniques.

Although uncertainty cannot be eliminated, its effects can be reduced to some extent. For example, the LOS concept helps to dampen the effects of uncertainty by presenting a range of service-measure results as being reasonably equivalent from a traveler's point of view. The use of a design hour, such as the 30th-highest hour of the year, also reduces uncertainty, as the variability of the design hour volume is much lower than the variability of individual hourly volumes throughout the year (*1*). Measured values will have more certainty than default values will, and multiple observations of a model input will have more certainty than a single observation. Sensitivity analyses—described later in this section—and other statistical techniques (*2*) can be used to test the impact of changes in model inputs on model outputs.

SOURCES OF UNCERTAINTY

Input Variables

HCM procedures and alternative tools typically require a variety of input data. Depending on the situation, there are up to three ways an analyst can provide these inputs. Listed in order of increasing uncertainty, they are (*a*) direct measurement, (*b*) locally generated default values, and (*c*) national default values suggested by the HCM or built into an alternative tool. Default values may not reflect spatial and temporal variability—national defaults to a greater extent than local defaults—because the mix of users and vehicles varies by facility and by time of day and because drivers' behavior varies depending on their familiarity with a facility and prevailing conditions. Direct measurements are subject only to temporal variability, as the measurement location's site-specific differences will be reflected in the observed values.

Day-to-day variability in traffic volume is a primary source of uncertainty in traffic analyses (*1, 3*). Future-year volumes are subject to higher degrees of uncertainty as one forecasts farther into the future because of unknowns about development patterns and timing, the timing of changes or additions to other parts of the transportation system, and changes in use of particular travel modes. Other input variables whose uncertainty has been studied in the literature are saturation flow rates (*4*), critical headways, and follow-up time (*5*).

Traffic volume variability from day to day and unknowns associated with future-year traffic volume forecasts are among the primary sources of uncertainty.

Model Accuracy

Model Development

Many HCM models are based on theoretically derived relationships, which include assumptions and contain parameters that must be calibrated on the basis of field data. Other HCM models are primarily statistical. The accuracy of these models can be described in terms of standard deviations, coefficients of determination of linear regression (R^2), and other statistical measures.

Only some of the older HCM models (i.e., those first appearing in the HCM2000 or earlier editions) have well-documented measures of uncertainty. On occasion, the Transportation Research Board's Committee on Highway Capacity and Quality of Service has exercised its judgment in modifying models to address illogical results (e.g., at boundary conditions) or to fill in gaps in small databases. In such cases, the "true" uncertainty of the entire model is virtually impossible to quantify. In contrast, most models developed for the HCM 2010 have documented measures of uncertainty. This information is provided in the original research reports for the HCM methodologies, which can be found in the Technical Reference Library in Volume 4.

Documentation of the uncertainty inherent in HCM models can be found in the models' original research reports, located in the Technical Reference Library in Volume 4.

Nested Algorithms

In many methodologies, the algorithm used to predict the final service measure relies on the output of another algorithm, which has its own uncertainty. Thus, the uncertainty of the final algorithm is compounded by the uncertainty in an input value derived from another algorithm.

In Chapter 12, Freeway Weaving Segments, for example, the prediction of weaving and nonweaving speeds depends on the free-flow speed and the total number of lane changes made by weaving and nonweaving vehicles. Each of these inputs is a prediction based on other algorithms, each having its own uncertainty. As additional examples, the urban street facility and freeway facility procedures are built on the results of underlying segment and (for urban streets) point models, the outputs of which have their own associated uncertainties.

Traveler Perception

Different people will have different levels of satisfaction for identical conditions.

The HCM 2010 introduces several traveler-perception-based models for estimating LOS for the bicycle, pedestrian, and transit modes. In addition, Chapter 17, Urban Street Segments, provides an alternative traveler-perception model for the automobile mode to help support multimodal analyses. These models produce estimates of the average LOS travelers would state for a particular system element and mode. However, people perceive conditions differently, which results in a range of responses (often covering the full LOS A to F range) for a given situation. As with other models, statistical measures can be used to describe the variation in the responses as well as the most likely response (*6*).

Additional Documentation

In addition to the uncertainty values given in the original research for HCM methods, the uncertainty of a number of current HCM models has been studied in the literature. These studies include unsignalized intersections (*5, 7, 8*), two-lane highways (*9*), and other uninterrupted-flow facilities (*10*).

Model Specification

A final potential source of uncertainty is an incomplete model specification, in which not all the factors that influence a model's result are reflected in the model's parameters. (An inaccurate specification, in which the wrong parameters are included in the model, also falls into this category.) The significant increase in the size of the HCM with each new edition, along with the complexity of its procedures, is due in part to users' desire for HCM models to account more fully for factors that influence the HCM's performance measures.

A more complex model is not necessarily a more accurate model.

However, a diminishing-returns principle also applies to model complexity. Each new variable added to a model brings with it uncertainty related to both the model's parameters and its input values. This additional complexity may not be warranted if the model's final output becomes more uncertain than it was before, even if the model appears to be more accurate by taking additional factors into account. Model complexity that leads to better decision making is justified; complexity that does not is best avoided (*11*).

SENSITIVITY ANALYSIS

One way to address the uncertainty inherent in a performance measurement estimate is to conduct a sensitivity analysis, in which key model inputs are individually varied over a range of reasonable values and the change in model outputs is observed. It is important for analysts to have a good understanding of

the sensitivity of model inputs, and special care should be taken in selecting appropriate values for particularly sensitive parameters. It is also important for analysts and decision makers to understand the sensitivity of model outputs (numerical values or the LOS letter grade) to changes in inputs, particularly volume forecasts, when they interpret the results of an analysis.

Sensitivity analysis is a useful technique for exploring how model outputs change in response to changes in model inputs.

Exhibit 7-1 illustrates a sensitivity analysis for selected inputs to the basic freeway segment method. A typical application would be a planning study for a future freeway, where not all the inputs are known exactly. The output being tested is the service volume (in vehicles per hour, veh/h) for LOS D (i.e., the highest volume that results in LOS D, given the other model inputs). The following inputs were held constant in all three examples:

- Lane width: 12 ft
- Percent trucks: 5%
- Driver population factor: 1.00
- Number of lanes per direction: 3
- Shoulder width: 6 ft
- Grade length: 1 mi

In each example, one of the following inputs was varied, while the other two were held constant. The varied input differs in each example:

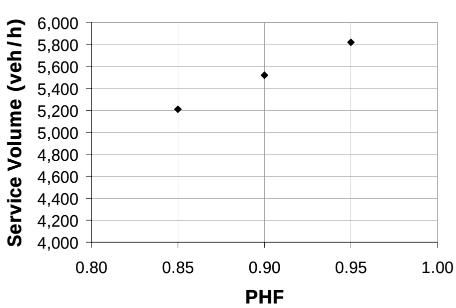
- Peak hour factor (PHF): 0.90, varied from 0.80 to 0.95 in Exhibit 7-1(a)
- Grade: 2%, varied from 1% to 6% in Exhibit 7-1(b)
- Total ramp density: 2 ramps/mi, varied from 1 to 4 ramps/mi in Exhibit 7-1(c)

Exhibit 7-1
Example Sensitivity Analysis for Selected Basic Freeway Segment Model Inputs

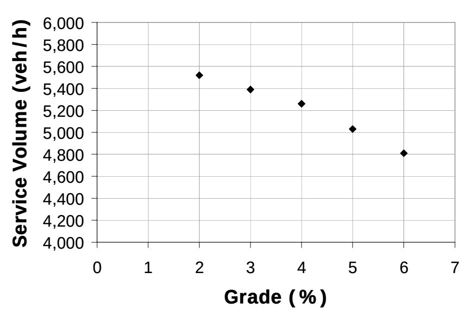
LIVE GRAPH
Click here to view

LIVE GRAPH
Click here to view

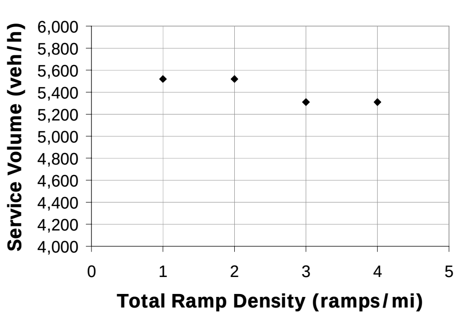
LIVE GRAPH
Click here to view



(a) Sensitivity to PHF



(b) Sensitivity to Grade



(c) Sensitivity to Total Ramp Density

If varying a single input parameter within its reasonable range results in a 0% to 10% change in the service-measure estimate, the model can be considered to have a low degree of sensitivity to that parameter. If a 10% to 20% change in the service-measure estimate results, the model can be considered moderately sensitive to that parameter, while if a change greater than 20% results, the model can be considered highly sensitive (*12*).

As shown in Exhibit 7-1(a) and Exhibit 7-1(b), LOS D service volumes for basic freeway segments are moderately sensitive to both PHF and grade across the reasonable ranges of values for those inputs, with the highest service volumes 12% and 15% higher than the lowest service volumes, respectively. Consequently, particular care should be taken to select appropriate values for these inputs.

When changing the total ramp density results in a free-flow speed estimate within the same 5-mi/h range used by a particular speed–flow curve, no change in the service volume result will be seen.

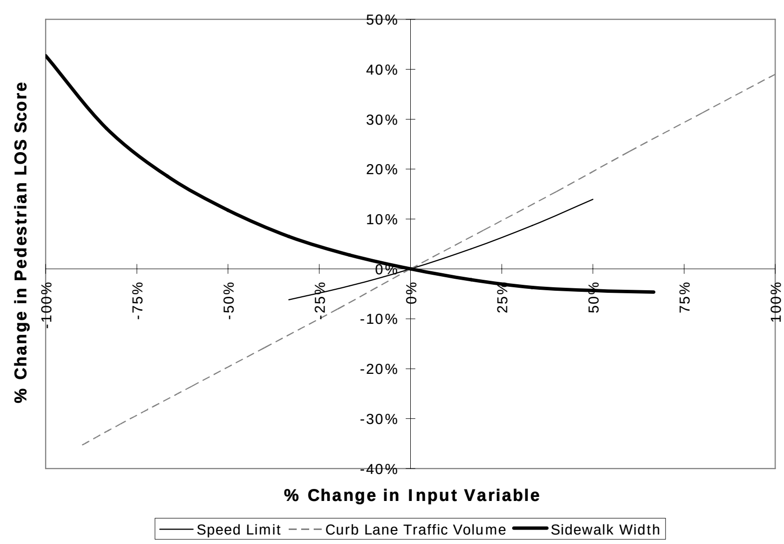
Exhibit 7-1(c) shows that LOS D service volumes have a low sensitivity to total ramp density, with just a 4% range in the output volumes. Therefore, it is less essential for the assumed average ramp density value to match the future condition closely.

Exhibit 7-2 shows an alternative way to display sensitivity information, based on the pedestrian link LOS score from Chapter 17, Urban Street Segments. In this example, the number of directional lanes (1), curb lane width (12 ft), and PHF (0.90) are held fixed, and it is assumed there is no bicycle lane, parking lane, or buffer between the sidewalk and the curb lane. The following inputs are varied one at time:

- Speed limit: 30 mi/h, varied from 20 to 45 mi/h;
- Curb lane traffic volume: 500 veh/h, varied from 50 to 1,000 veh/h; and
- Sidewalk width: 6 ft, varied from 0 to 10 ft.

Exhibit 7-2
Example Sensitivity Analysis of Urban Street Link Pedestrian LOS Score

LIVE GRAPH
Click here to view

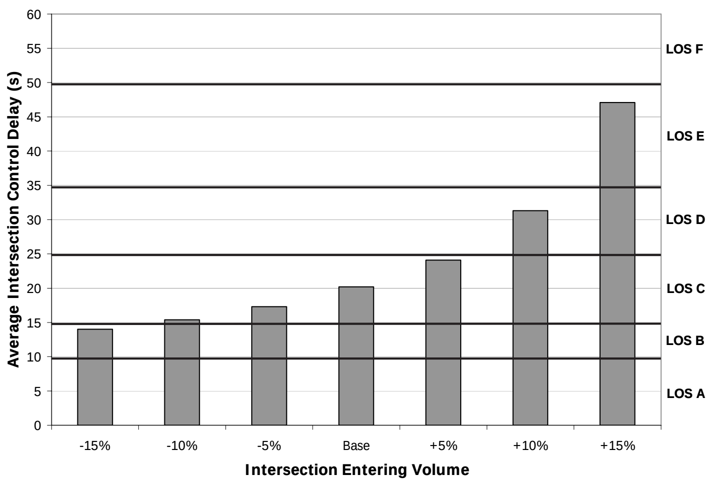


The pedestrian LOS score is relatively insensitive to speed limit, moderately sensitive to sidewalk width (except when a sidewalk is not present), and highly sensitive to curb lane traffic volume. This kind of presentation works best when

typical values for the input variables to be tested lie near the middle of their range rather than at or near one of the extremes.

Exhibit 7-3 shows an example of testing the sensitivity of control delay, and the corresponding LOS result, at an all-way STOP-controlled intersection, by varying the demand volumes used in the analysis. In the exhibit, the base volume entering the intersection on all approaches is varied within a ±15% range in 5% increments. This kind of sensitivity analysis is particularly useful in working with forecasts of volume that have a relatively high degree of uncertainty associated with them.

Exhibit 7-3
Example Sensitivity Analysis of All-Way STOP-Control Model Outputs Based on Varying Volume Inputs



Note: Values used in the calculation are four-legged intersection with one lane on each approach, PHF = 0.90, and 2% heavy vehicles. Base volumes are 210 through vehicles, 35 left-turning vehicles, and 35 right-turning vehicles on each approach.

Depending on the model and the specifics of the situation being modeled, relatively small changes in model inputs can have relatively large impacts on model outputs.

As shown in Exhibit 7-3, under the base volume forecast, the intersection is forecast to operate at LOS C. If future traffic volumes are lower than forecast or as much as 5% higher than forecast, the intersection will still operate at LOS C or better. If future traffic volumes are 10% higher than forecast, the intersection will operate at LOS D; if traffic volumes are 15% higher than forecast, the intersection will operate at LOS E. If the jurisdiction's operations standard for the intersection is LOS E or better, it could be reasonably concluded that the intersection would still operate acceptably even if higher volumes than forecast were to occur. However, if the standard was LOS D or better, a closer look at the conservativeness of the volume forecasts might be needed to conclude that the intersection would operate acceptably.

ACCURACY AND PRECISION

Overview

Accuracy and precision are independent but complementary concepts. *Accuracy* relates to achieving a correct answer, while *precision* relates to the size of the estimation range of the parameter in question. As an example of accuracy, consider a method that is applied to estimate a performance measure. If the performance measure is delay, an accurate method would provide an estimate closely approximating the actual delay that occurs under field conditions. The

precision of such an estimate is the range that would be acceptable from an analyst's perspective in providing an accurate estimate. Such a range might be expressed as the central value for the estimated delay plus or minus several seconds.

In general, the inputs used by HCM methodologies come from field data or estimates of future conditions. In either case, these inputs can be expected to be accurate to only within 5% or 10% of the true value. Thus, the computations performed with these inputs cannot be expected to be extremely accurate, and the final results must be considered as estimates that are accurate and precise only within the limits of the inputs used.

HCM users should be aware of the limitations of the accuracy and precision of the methodologies in the manual. Such awareness will help users interpret the results of an analysis and use the results to make a decision about the design or operation of a transportation facility.

Calculation Precision Versus Display Precision

Precision in calculation differs from precision in presenting final results.

The extensive use of personal computers has allowed calculations of capacity and LOS to be carried to a large number of digits to the right of the decimal point. Because of this ease of calculation, there is a need to state clearly that the final result of calculations done manually and carried to the suggested number of significant figures might be slightly different from the result of calculations performed on a computer.

Implied Precision of Results

The typical interpretation given to a value such as 2.0 is that the value is in a precision range of two significant figures and that results from calculations should be rounded to this level of precision. The actual computational result would have been in the range of 1.95 to 2.04 by standard rounding conventions. Occasionally, particularly in the running text of the HCM, editorial flexibility allows a zero to be dropped from the number of digits. In most cases, however, the number of the digits to the right of the decimal point does imply that a factor or numerical value has been calculated to that level of precision.

AVERAGE VALUES

Unless specifically noted otherwise, HCM performance measure estimates are average (mean) values.

Unless otherwise noted or defined, numerical values are mean values for the given parameter. Thus, a measure of speed or delay is the mean value for the population of vehicles (or persons) being analyzed. Similarly, a lane width for two or more lanes is the mean (average) width of the lanes. The word "average" or "mean" is only occasionally carried along in the text or exhibits to reinforce this otherwise implicit fact. LOS threshold values, adjustment factors used in computations, and calculated values of performance measures are assumed to represent conditions that have a reasonable expectation of being observed regularly in North America, as opposed to the most extreme condition that might be encountered.

3. DEFINING AND COMPUTING UNIFORM PERFORMANCE MEASURES

The exact definition of performance measures poses an important question, particularly when a comparison of performance measures produced by different tools is to be made. Definitions and computational methods are especially important when the LOS must be inferred from another performance measure obtained by alternative methods and applied to the thresholds presented in the HCM's procedural chapters. Often, a performance measure is given the same name in various tools, but its definition and interpretation differ.

This section identifies the performance measures commonly reported by the procedures given in Volumes 2 and 3. The concept of developing these measures from an analysis of the individual vehicle trajectories produced by microsimulation tools is introduced. The most important measures are discussed in terms of uniform definitions and methods of computation that will promote comparability among different tools. More detailed procedures for developing performance measures from individual vehicle trajectories are presented in Chapter 24, Concepts: Supplemental.

PERFORMANCE MEASURES REPORTED BY HCM PROCEDURES

The key performance measures associated with each of the procedural chapters in Volumes 2 and 3 were summarized in Chapter 6, HCM and Alternative Analysis Tools. The applicability of these procedures and alternative tools was indicated for each type of facility. Exhibit 7-4 includes all the performance measures identified in Chapter 6. The service measures that determine LOS for each system element are also identified. In this section, the key performance measures are presented in terms of their definitions and computational procedures. The potential for the development of uniform performance measures from alternative tools is presented in a later section.

Exhibit 7-4
Performance Measures Reported by HCM Procedures

Chapter	Density	Speed	*v/c* Ratio*	Travel Time	Control Delay	Queue	Other Measures
10. Freeway Facilities	**Yes**	Yes	Yes	Yes		Yes	*a*
11. Basic Freeway Segments	**Yes**	Yes	Yes				
12. Freeway Weaving Segments	**Yes**	Yes	Yes				*b*
13. Freeway Merge and Diverge Segments	**Yes**	Yes	Yes				
14. Multilane Highways	**Yes**	Yes	Yes				
15. Two-Lane Highways		**Yes**	Yes				*c*
16. Urban Street Facilities		Yes		Yes	Yes		*d*
17. Urban Street Segments		Yes		Yes	Yes		*d*
18. Signalized Intersections			Yes		**Yes**	Yes	
19. TWSC Intersections			Yes		**Yes**	Yes	
20. AWSC Intersections			Yes		**Yes**	Yes	
21. Roundabouts			Yes		**Yes**	Yes	
22. Interchange Ramp Terminals			Yes		**Yes**	Yes	
23. Off-Street Pedestrian/Bicycle Facilities							*e*

Notes: *v/c*, volume/capacity ratio; TWSC, two-way STOP-controlled; AWSC, all-way STOP-controlled; **bold** text indicates service measures.
[*] A *v/c* ratio greater than 1.00 is often used to define LOS F conditions.
[a] Vehicle miles, vehicle hours.
[b] Weaving speed, nonweaving speed.
[c] **Percent time spent following.**
[d] Stop rate, running time.
[e] **Meeting and passing events.**

Speed-Related Measures

Speeds are reported in several chapters of this manual:

- *Chapter 10, Freeway Facilities*, uses the average speeds computed by the other freeway procedural chapters when all segments are undersaturated. When demand exceeds capacity, the speeds on the affected segments are modified to account for the effects of slower-moving queues.
- *Chapter 11, Basic Freeway Segments*, estimates the average speed on the basis of the free-flow speed and demand volume by using empirically derived relationships.
- *Chapter 12, Freeway Weaving Segments*, estimates the average speed as a composite of the speeds of weaving and nonweaving vehicles on the basis of free-flow speed, demand volumes, and geometric characteristics. The computational procedures for estimating the actual speeds are based on the nature of the weaving segment and the origin–destination matrix of traffic entering and leaving the segment. The speed estimation processes are substantially more complex in weaving segments than in basic freeway segments.
- *Chapter 13, Freeway Merge and Diverge Segments*, estimates the average speed of vehicles across all lanes as well as the average speeds in the lanes adjacent to the ramp. The computations are based on empirical relationships specifically derived for merge and diverge segments.
- *Chapter 15, Two-Lane Highways*, treats the average travel speed (ATS) on certain classes of highways as one determinant of LOS. The ATS is determined as an empirical function of free-flow speed, demand flow rates, proportion of heavy vehicles, and grades.
- *Chapter 16, Urban Street Facilities*, uses "travel speed as a percentage of base free-flow speed" to determine LOS.
- *Chapter 17, Urban Street Segments*, also uses speed to determine LOS. The average speed is computed by dividing the segment length by the average travel time. The average travel time is determined as the sum of three components:

  1. Time to traverse the link at the running speed, which is computed as a function of the free-flow speed, demand flow rate, and geometric factors;
  2. Control delay due to the traffic control device at the end of the segment; and
  3. Midblock delay due to access points.

  The average speed applies only to arterial through vehicles and not to the traffic stream as a whole.

Queue-Related Measures

Queue measures are defined and computed for both interrupted- and uninterrupted-flow facilities. Queues may be defined in terms of the number of vehicles contained in the queue or the distance of the last vehicle in the queue from the end of the segment (i.e., back of queue or BOQ). The probability of the BOQ reaching a specified point where it will cause problems is of most interest to the analyst. For most purposes, the BOQ is therefore a more useful measure than the number of vehicles in the queue.

Queue measures are reported by the following procedures in this manual:

- *Chapter 10, Freeway Facilities*: Queuing on freeway facilities is generally the result of oversaturation caused by excessive demand or by bottlenecks. As such, it is treated deterministically in Chapter 10 by an input–output model that tracks demand volumes and actual volume served through the bottleneck. The propagation and dissipation of freeway queues are estimated from a linear deterministic queuing model. The shock wave speed at which queues grow and shrink is calculated from the jam density and the flow and density at capacity. Residual demand is processed in subsequent time intervals as demand levels drop or the bottleneck capacity increases. Generally, a drop in demand results in a queue that clears from the back, while an increase in bottleneck capacity results in a forward-clearing queue. The spatial extent of the queue is calculated from the number of queued vehicles and the storage space on the facility (i.e., the length and number of lanes). The temporal duration of the queue is a function of demand patterns and bottleneck capacity. The presence of a queue on a given segment also affects the rate at which vehicles can flow into the next segment. The volume arriving in downstream segments may therefore be less than the demand volume. Downstream segments with demand volumes greater than capacity may turn out to be hidden bottlenecks if a more severe upstream bottleneck meters the volume served.
- *Chapter 18, Signalized Intersections*: The cyclical maximum BOQ is computed on the basis of a queue accumulation and discharge model with a correction applied to account for acceleration and deceleration. Random arrivals and oversaturated conditions are accommodated by correction terms in the model. The computational details are provided in Chapter 31, Signalized Intersections: Supplemental. The maximum number of vehicles in the queue occurs at the end of the red phase in each cycle. The maximum BOQ extends into the subsequent green phase because vehicles continue to join the BOQ until it is fully dissipated. The measure reported for signalized approaches is the average BOQ. Percentile values are also reported.
- *Chapters 19 to 21, unsignalized intersections*: The 95th percentile BOQ is computed by deterministic equations as a function of demand volume, capacity, and analysis period length.

Stop-Related Measures

Stop-related measures are of interest to analysts because of their comfort, convenience, cost, and safety implications. An estimate of the number of stops on a signalized approach is reported by the signalized intersection analysis procedure described in Chapter 18, with details given in Chapter 31. Chapter 17, Urban Street Segments, incorporates the stops at the signal into a "stops per mile" rate for each segment. Other chapters do not report the number of stops. Most alternative tools based on both deterministic and simulation models produce an estimate of the number of stops for a variety of system elements by using the tools' own definitions, and most tools allow user-specified values for the parameters that define when a vehicle is stopped.

The Chapter 18 procedure defines a "partial" stop as one in which a vehicle slows as it approaches the BOQ but does not come to a full stop. Some alternative tools, both deterministic and simulation based, consider a partial stop to be a later stop after the first full stop.

Delay-Related Measures

The definition and computation of delay vary widely among tools.

Because of multiple definitions and thresholds, delay is one of the most difficult measures to compare among traffic analysis tools. Delay measures are reported by the same chapters in this manual that report queue measures:

- *Chapter 10, Freeway Facilities*: Delay on a freeway facility that is globally undersaturated is calculated from the sum of the delay on all individual segments. The segment delays are calculated from the travel time difference between the segment at free-flow speed and the segment at the calculated operational space mean speed. For undersaturated flow conditions, the segment space mean speed is calculated from the segment-specific methodologies in Chapters 11 to 13. For oversaturated flow, the segment speed is estimated from the prevailing density on the segment. The travel time difference is multiplied by the number of vehicles in a segment during each time period to obtain the total vehicle hours of delay per segment and per time period. The total vehicle hours of delay on the facility for each time period and for the entire analysis are obtained by summation.
- *Chapter 18, Signalized Intersections*: The LOS estimation is based on control delay. Control delay is computed on the basis of an incremental queue analysis technique by using a queue accumulation and discharge model. Random arrivals and oversaturated conditions are accommodated by correction terms in the model. A separate correction is applied to account for an initial queue left from a previous interval. The details of the computation are provided in Chapter 31, Signalized Intersections: Supplemental.
- *Chapters 19 to 21, unsignalized intersections*: The LOS estimation is based on control delay. The control delay is computed by deterministic equations as a function of demand volume, capacity, and analysis period length. The LOS thresholds for unsignalized intersections are different from those for signalized intersections.

Density-Related Measures

Density is expressed in terms of vehicles per mile per lane and is generally recognized as an unambiguous indicator of congestion. Density is used as the determinant of LOS A through E for freeway and multilane highway segments. It is conceptually easy to define and estimate, but the question is how to apply density to the right section of roadway over the right period of time.

The procedures for different types of freeway segments follow a density estimation process that is specific to each segment type:

- *Chapter 10, Freeway Facilities*, determines density for undersaturated conditions by applying the procedures given in Chapters 11 to 13. When queuing occurs as a result of oversaturation caused by excessive demand or by bottlenecks, the density is determined by the queue tracking procedures described previously for freeway facilities.
- *Chapter 11, Basic Freeway Segments*, determines speeds and demand flow rates that are adjusted for a variety of geometric and operational conditions. The segment density is computed by dividing the adjusted flow rate by the estimated speed. Empirical relationships are used throughout the chapter for computations and adjustments.
- *Chapter 12, Freeway Weaving Segments*, also determines density by dividing the adjusted demand flow rate by the estimated speed. The speed estimation process was described previously.
- *Chapter 13, Freeway Merge and Diverge Segments*, bases the LOS assessment on the density in the two lanes adjacent to the ramp lanes. The density is estimated directly by using empirically derived relationships that depend on the ramp and freeway (Lanes 1 and 2) volumes and the length of the acceleration or deceleration lane. Several operational and geometric factors affect the computations.

USE OF VEHICLE TRAJECTORY ANALYSIS IN COMPARING PERFORMANCE MEASURES

This section explores the use of vehicle trajectory analysis to define and estimate consistent performance measures. It first introduces the mathematical properties of trajectories as an extension of the visual properties. It identifies the types of analyses that can be performed and provides examples that illustrate how trajectory analysis can be applied. A later section identifies the performance measures that can be computed from individual vehicle trajectories and explores their compatibility with the performance measures estimated by the HCM's computational procedures. Specific trajectory analysis procedures by which consistent performance measures can be estimated are presented in Chapter 24, Concepts: Supplemental.

The concept of individual vehicle trajectory analysis was introduced in Chapter 4, Traffic Flow and Capacity Concepts. The chapter indicated that a growing school of thought suggests that a comparison of results between traffic analysis tools and methods is possible only through an analysis of vehicle trajectories as the "lowest common denominator." Trajectory-based performance measures can be made consistent with HCM definitions, with field measurement

techniques, and with each other. Examples of vehicle trajectory plots were shown in Chapter 4 to illustrate the visual properties of vehicle trajectories.

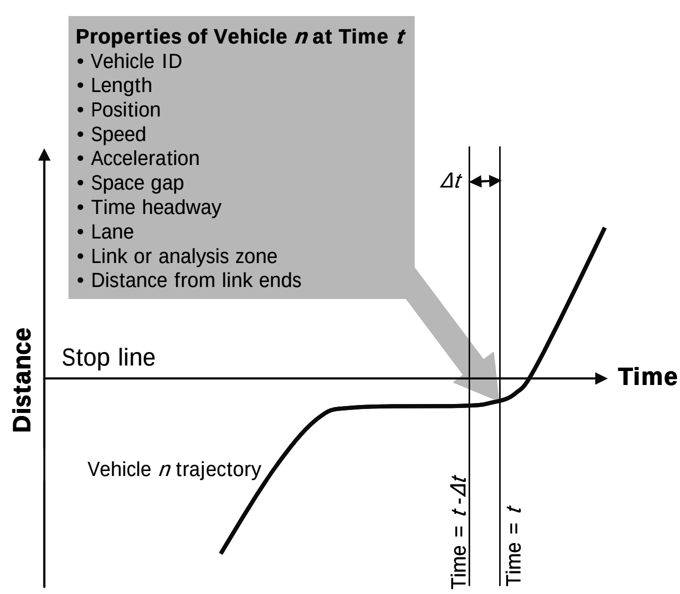
Mathematical Properties of Vehicle Trajectory

While the trajectory plots presented in Chapter 4 provide a good visual insight into operations, they do not support any quantitative assessments. To develop performance measures from vehicle trajectories, it is necessary to represent them mathematically and not just visually. A mathematical representation requires developing a set of properties that are associated with each vehicle at specific points in time and space.

Exhibit 7-5 shows the trajectory of a single vehicle through a signal. At each point in time, a number of properties may be determined. The trajectory for the vehicle is quantified through a list of the properties of vehicle *n* at each point in time. One important parameter in the quantification of trajectories is the time increment between sampling points, represented in Exhibit 7-5 as Δt. Time increments in typical simulation tools currently range from 0.1 to 1.0 s. Smaller values are gaining acceptance within the simulation modeling community because of their ability to represent traffic flow with greater fidelity.

Many properties can be associated with a specific vehicle at a point in time. Some properties are required for the accurate determination of performance measures from trajectories. Others are used for different purposes such as safety analysis. The important properties for estimating consistent performance measures are indicated in Exhibit 7-5.

Exhibit 7-5
Mathematical Properties of Vehicle Trajectories



Longitudinal and Spatial Analysis

It is necessary at the outset to distinguish between longitudinal and spatial analysis of vehicle trajectories. Longitudinal analysis involves following the position of vehicles as they traverse a segment. The measures determined by this type of analysis include delay-related measures of various types and stop-related

measures. Driver comfort and safety measures may also be determined by longitudinal analysis, but these measures are beyond the scope of the HCM.

Spatial analysis, on the other hand, involves considering all the vehicles on a segment at a specific time step. The two principal spatial measures include density and queue lengths. Both types of analysis are examined here.

Limitations of Vehicle Trajectory Analysis

The procedures described here and in Chapter 24 are intended to produce performance measures from vehicle trajectories that are based on the definitions of traffic parameters given in this manual to promote uniformity of reporting among different simulation tools. The results should improve the acceptance of simulation tools for highway capacity and LOS analysis. However, the term "HCM-compatible" does not suggest that the numerical values of measures produced by a simulation tool will be identical to those from the HCM or to those from other simulation tools. Several factors must be considered.

Traffic Modeling Differences

The trajectory information is produced by the simulation model. Each simulation tool has its own models of driver behavior that differ from those used by other tools. It is not practical or desirable to prescribe simulation modeling details in this document. Developers continually strive to improve the realism of their products to gain a competitive advantage in the market. The Next Generation Simulation Program (*13*) has had some success in developing core algorithms to be shared by simulation developers, but the notion of a universal simulation model is not a practical objective.

Approximations in Trajectory Analysis

It was pointed out in Chapter 4 that all performance measures reported by deterministic models, simulation models, and field observations represent an approximate assessment of field conditions. The need for approximations in trajectory analysis to promote uniform reporting is explored in more detail in Chapter 24. One problem is that the procedures prescribed in this manual introduce approximations that cannot be replicated in simulation because of conceptual differences and model structure.

Differences That Are Unrelated to Trajectory Analysis

The use of vehicle trajectories addresses some, but not all, of the sources of difference in the definition of performance measures. For example, the temporal and spatial boundaries of an analysis tend to be defined differently by different tools. It is important to HCM compatibility that the definitions and guidelines presented in this manual be used in conducting simulation analyses.

Examples of Vehicle Trajectory Data

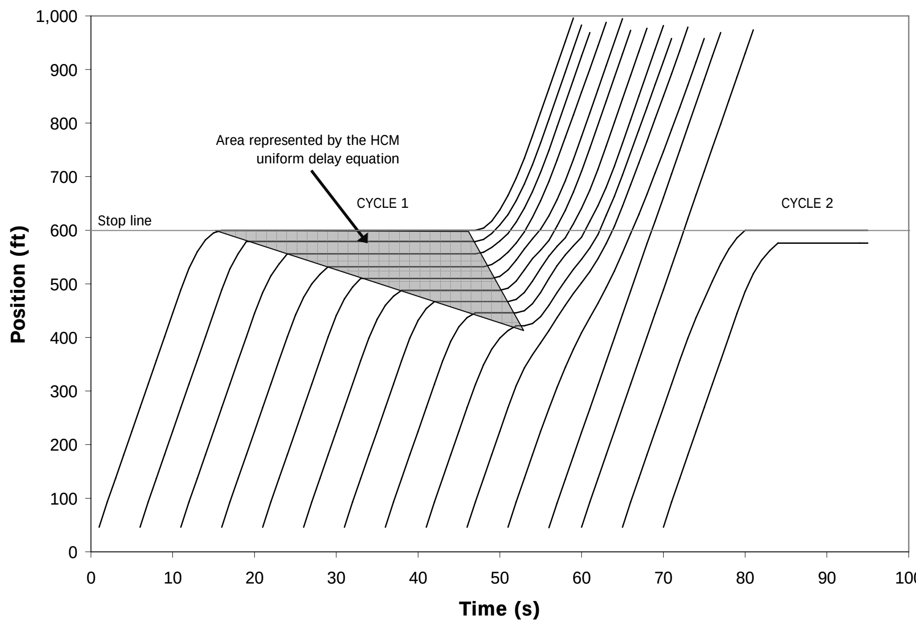
Simulation tools propagate vehicles through a roadway segment by periodically updating and keeping track of the trajectory properties that are maintained internally within the traffic flow model. Several examples of the analysis of vehicle trajectories on both interrupted- and uninterrupted-flow

facilities are provided in Chapter 24. The examples demonstrate the complexities that can arise in certain situations, especially when demand exceeds capacity.

Two examples included in Chapter 24 are presented here to illustrate how vehicle trajectories can be obtained from simulation tools. The first is shown in Exhibit 7-6, which presents the simplest possible case, involving an approach with only one lane. The simulation parameters were constrained to remove all randomness in the arrival and departure characteristics. While this situation might appear to be somewhat trivial, it is the basis of the signalized intersection delay analysis procedure summarized in Chapter 18 and described in more detail in Chapter 31.

Exhibit 7-6
Trajectory Plot for Uniform Arrivals and Departures

LIVE GRAPH
Click here to view



The trajectories may be analyzed longitudinally to produce estimates of delays and stops. They may also be analyzed spatially to produce instantaneous queue length estimates.

A more complex situation is depicted in Exhibit 7-7, which illustrates the vehicle trajectories associated with queue backup from a downstream signal. The randomness of arrivals and departures was restored to this case.

The important difference from the simple case presented in Exhibit 7-6 is that backup into a specific segment from a downstream segment is not covered by the signalized intersection analysis procedures in Chapters 18 and 31. The performance measures may be estimated by trajectory analysis. The purpose of the procedures described in Chapter 24 is to promote definitions and computational algorithms that can be made consistent among different simulation tools.

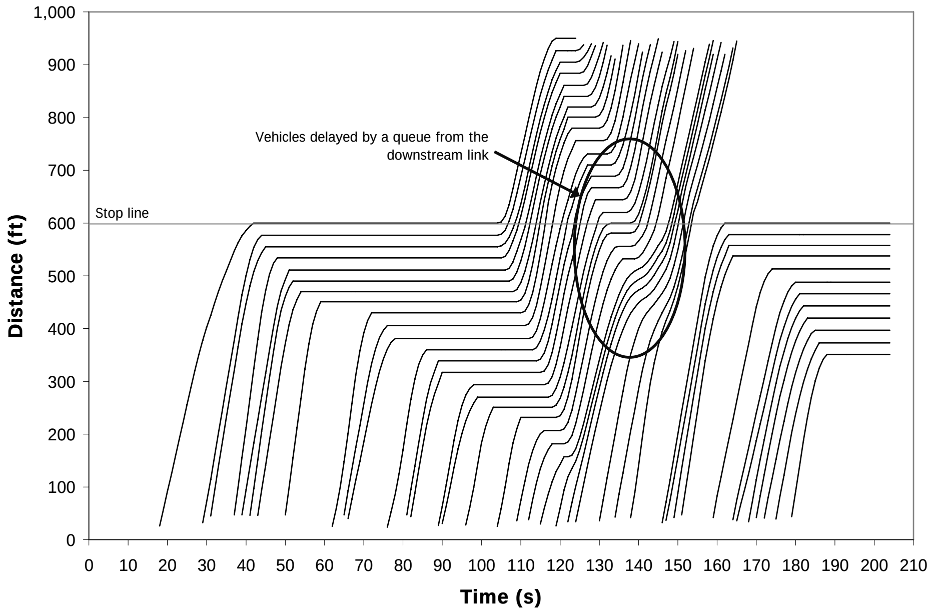


Exhibit 7-7
Queue Backup from a Downstream Signal

LIVE GRAPH
Click here to view

REQUIREMENTS FOR COMPUTING PERFORMANCE MEASURES BY VEHICLE TRAJECTORY ANALYSIS

Most performance measures reported by the procedures in this manual are also reported by simulation tools. This section identifies the general requirements for computing measures from simulation by using individual vehicle trajectories to achieve comparability between traffic analysis tools. More detailed procedures are presented in Chapter 24.

General Trajectory Analysis Guidelines

The following general guidelines apply to trajectory analysis procedures.

1. The trajectory analysis procedures are limited to the analysis of trajectories produced by the traffic flow model of each simulation tool. The nature of the procedures does not suggest the need for developers to change their driver behavior or traffic flow modeling logic.
2. If the procedures for estimating a particular measure cannot be satisfactorily defined to permit a valid comparison between the HCM and other modeling approaches, then such comparisons should not be made.
3. All performance measures that accrue over time and space shall be assigned to the link and time interval in which they occur. Subtle complexities make it impractical to do otherwise. For example, the root cause of a specific delay might not be within the link or the immediate downstream link. In fact, the delay might be secondary to a problem at some distant location in the network and in a different time interval.
4. The analyst must understand that the spatial and temporal boundaries of the analysis domain must include a period that is free of congestion on all sides. This principle is also stated in Chapter 10 for analysis of freeway facilities and in Chapter 18 for multiperiod signalized intersection analysis. To ensure that delays to vehicles that are denied entry to the

system during a given period are properly recognized, it might be necessary to create fictitious links outside of the physical network to hold such vehicles. A more detailed discussion of spatial and temporal boundaries is provided later in this chapter.

5. It is important to ensure that the network has been properly initialized or "seeded" before trajectory analysis is performed. When setting and applying the warm-up periods, simulation tools typically start with an empty network and introduce vehicles until the vehicular content of the network stabilizes. Trajectory analysis should not begin until stability has been achieved. If the simulation period begins with oversaturated conditions, stability may never be achieved. See the discussion later in this chapter on temporal and spatial boundaries.

Speed- and Travel Time–Related Measures

Speed and travel time are treated together because, at least for segment values, they are closely related. The average speed of a vehicle traversing a segment may be determined by dividing the segment length by the travel time.

Macroscopic segment travel time estimation does not require a detailed trajectory analysis. The travel time for an individual vehicle may be computed for a given segment by subtracting the time when the vehicle entered the segment from the time when it left the segment. The average travel time may be computed as the mean of the individual travel times; however, this technique is valid only for complete trips (i.e., those that have entered and left the segment).

The space mean speed for all vehicles within the segment during the time period may be estimated by dividing the number of vehicle miles of travel by the number of vehicle hours of travel time. The total vehicle miles and vehicle hours may be accumulated by including all the vehicles and time steps in the analysis domain. See the discussion later in this chapter on spatial and temporal boundaries.

Queue-Related Measures

Because of their microscopic nature, simulation tools have the potential to produce useful measures of queuing that are beyond the limits of those described in the procedural chapters of this manual. However, the queue-related performance measures are difficult to compare with those derived from HCM procedures. No such comparisons should be attempted without a detailed knowledge of the queue definitions and computations of a specific tool. With consistent definitions, more uniform queue measures could be obtained from simulation tools.

Queued State

What defines the entry to and exit from a queue? Several definitions are applied by different tools for this purpose. The definition given in Chapter 31 for purposes of field observations states the following:

A vehicle is considered as having joined the queue when it approaches within one car length of a stopped vehicle and is itself about to stop. This definition is

used because of the difficulty of keeping precise track of the moment when a vehicle comes to a stop.

The Chapter 31 definition of the exit from a queue, also intended for application to field studies, is more complex and offers some interesting challenges for implementation in both deterministic and simulation models. As a practical approximation, a vehicle should be considered to have left the queue when it has left the link in which it entered the queue. When a queue extends the full length of a link, a vehicle should be considered to enter the queue at the time it enters the link. Other conditions, such as a lane change to escape a queue, might also signal the exit from a queue. These conditions are discussed in Chapter 24.

Queue Length

The estimation of queue length is generally required to determine whether a queue has reached the point where it will interfere with other traffic movements. Queue length computations are applied at a macroscopic level by the procedures described in this manual. Simulation models, on the other hand, are able to establish the instantaneous BOQ at each point in time. The question is how to process the instantaneous values in a manner that will produce meaningful results.

Queue length analysis by simulation must be treated differently for different conditions. There are three cases to consider:

1. *Undersaturated noncyclical operation*, typical of the operation with isolated two-way STOP control: In this case, the queue accumulation and discharge follow a more or less random pattern. The Chapter 19 procedure estimates the 95th percentile queue length on the basis of a deterministic average queue length modified by a term that accounts for random arrivals. This process could be approximated in trajectory analysis by establishing a distribution of instantaneous queue lengths by time step. The 95th percentile queue length could be determined from that distribution.
2. *Undersaturated cyclical operation*, typical of the operation at a traffic signal: In this case, a maximum BOQ is associated with each cycle. The maximum BOQ in each cycle represents one observation for statistical analysis purposes. The use of a distribution of instantaneous values is not appropriate here because the queue accumulation and discharge are much more systematic than random. Including instantaneous queue lengths that occur when the queue is expected to be zero (i.e., at the end of the green) would underestimate the measure of interest, which is the peak queue length. With a sufficient number of cycles, a distribution of peak queue lengths with a mean value and a standard deviation could be established. The probability of queue backup to any point could then be estimated from this distribution.
3. *Oversaturated operation*, either cyclical or noncyclical: When demand exceeds the capacity of an approach or system element, the queue will grow indefinitely. For purposes of simulation, the measure of interest is

the residual BOQ at the end of the simulated interval and the effect of the queue on upstream segments. These considerations are especially important in multiperiod analyses.

The undersaturated condition might include brief periods of queue buildup and discharge as long as continuous buildup and residual queues do not occur.

Stop-Related Measures

Most alternative tools based on both deterministic and simulation models produce an estimate of the number of stops by their own definition, and most allow user-specified values for the parameters that establish the beginning and end of a stop. Stop-related measures are of interest to analysts because of their comfort, convenience, cost, and safety implications.

Definition of the Stopped State

The definition of when a vehicle is stopped has the same two elements as the definition of when it is queued—that is, when does the stop begin and when does it end? Speed thresholds are often used to determine when a vehicle is stopped. The only nonarbitrary threshold for this purpose is zero. Practical considerations suggest, however, that simulation modeling algorithms that deal with stopping would be more stable if a near-zero speed were used instead. A speed of 5 mi/h was applied in other chapters for determining when a vehicle has stopped.

There are two different modeling purposes for releasing a vehicle from the stopped state:

- To terminate the accumulation of stopped delay, and
- To enable the accumulation of subsequent stops.

The first condition is easier to deal with in the trajectory analysis. When the vehicle is no longer stopped, it should no longer accumulate stopped delay. The logical speed threshold for this condition is the same speed threshold that established the beginning of the stop.

Estimating the Number of Stops

The accumulation of multiple stops poses more problems and generally relies on arbitrary thresholds that vary among different tools. The main problem with multiple stops is that, after the first stop, later stops take place from a lower speed and therefore have a less adverse effect on driver comfort, operating costs, and safety. For signalized approaches, some tools apply a "probability of stopping" model in which the maximum probability is 100% and, therefore, the maximum number of stops is 1.0 on any approach. Other tools model subsequent stops based on the release from the stopped state when the vehicle reaches an arbitrary threshold speed, often around 15 mi/h.

While the number of stops is an important performance measure, it is difficult to compare the values produced by different tools. Such comparisons should not be attempted without adequate knowledge of the definitions and parameters that are used by a specific tool.

Delay-Related Measures

Practically all traffic analysis tools produce a performance measure called "delay," but tools vary widely in the definition and computation of delay. This discussion attempts to establish consistent definitions for delay.

Delay Definitions

Delay is generally defined as the excess time spent on a road segment compared with the time at a target speed that represents a zero-delay condition. The target speed is the speed at which a specific driver prefers to drive. Different tools have different definitions of target speed. Some are driver and vehicle specific, taking into account driver aggressiveness and roadway characteristics. Because target speed is a function of individual driver behavior, there will be some differences in the method of computation, especially if the target speed is different for each vehicle. For tools that require a user-specified free-flow speed as an input, the methodology presented in the procedural chapters of this manual should be used to determine the free-flow speed.

The time a vehicle actually spends on a segment is easy to determine from its trajectory. On the other hand, the target time is subject to a number of definitions:

- *Travel time at ideal speed*: usually the free-flow speed.
- *Travel time at the individual vehicle's target speed*: a function of the free-flow speed, prevailing roadway and traffic conditions, and the driver's characteristics.
- *Travel time at 10 mi/h below speed limit*: used by some agencies to determine whether a trip is "on time" for travel time reliability reporting. This measure defines "on-time delay."
- *Travel time at a specified travel time index*: The travel time index is the ratio of actual travel time to ideal travel time. It is used primarily for reporting congestion in nationwide mobility monitoring. A travel time index of 1.5 is sometimes taken as an indication of congestion. This measure defines congestion delay. It is intended to be an indicator of the need for roadway improvements.
- *Travel time without traffic control*: This measure defines control delay. Unlike the previous measures, which are applied to an entire segment, control delay is applied only to the portion of the segment where a queue is present. Control delay is a subset of segment delay because it does not include the delays caused by traffic interactions upstream of the queue. The definition applies uniformly to all types of control, including signals, stop signs, and roundabouts.

In all cases, a lower limit of zero must be imposed when the actual travel time is shorter than the reference time.

Aggregated Delay Versus Unit Delay

It is important to note the difference between aggregated delay, usually expressed in vehicle hours, and unit delay, usually expressed in seconds per

vehicle. Aggregated delay is generally used to assess the operating costs associated with a candidate treatment, because an economic value can be assigned to a vehicle hour of delay. Unit delays are associated with driver perception of the LOS on a facility. For these two definitions to be dimensionally consistent, the unit delays must actually be expressed in vehicle seconds per vehicle. It is common practice, however, to shorten the definition to seconds per vehicle to promote public understanding.

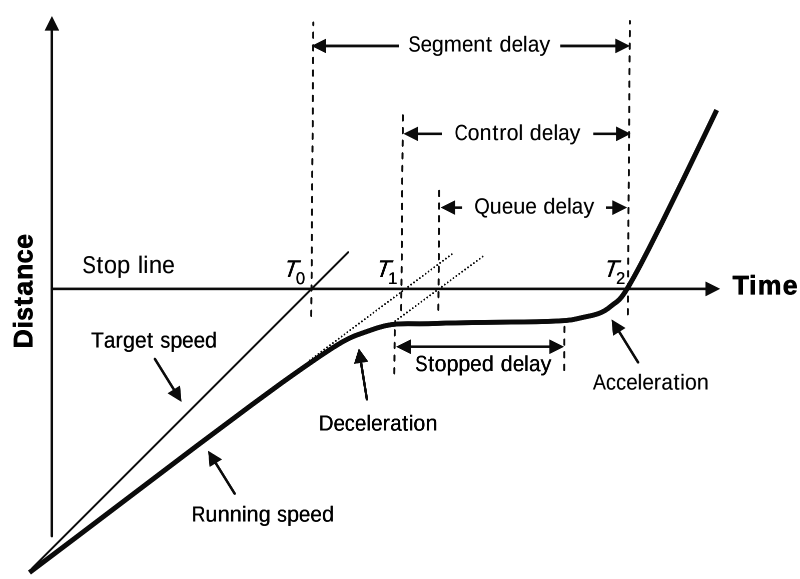
Representation of Delay by Vehicle Trajectories

Several delay definitions were presented previously. These definitions may be interpreted in terms of vehicle trajectories based on longitudinal trajectory analysis. In all cases, the delay is determined for each time step and accumulated over the entire time the vehicle was in a specified segment.

Exhibit 7-8 illustrates the various ways delay may be defined. Three points are defined in this figure.

- T_0, the time at which a vehicle would have arrived at the stop line if it had been traveling at the target speed;
- T_1, the time at which a vehicle would have arrived at the stop line if it had been traveling at the running speed, which is generally less than the target speed because of traffic interactions; and
- T_2, the time at which a vehicle is discharged at the stop line.

Exhibit 7-8
Definition of Delay Terms in Time and Space



The delay measures defined in terms of the time differences shown in Exhibit 7-8 include the following:

- *Control delay*: defined as $T_2 - T_1$. This delay definition is the one used by the procedure for assessing LOS at controlled intersections and roundabouts.

- *Segment delay*, defined as $T_2 - T_0$. This definition is more commonly used by simulation tools. It reflects the delay experienced by each vehicle since it left the upstream node (usually another signal). Segment delay includes control delay plus all other delay due to traffic interactions.

In addition to these two precisely defined delay terms, two other delay definitions are shown in Exhibit 7-8 that are based on more complex properties of the vehicle trajectories:

- *Stopped delay*, which reflects the amount of time a vehicle was actually stopped. The beginning and end of a stop are generally based on speed thresholds, which may differ among tools. In some cases, the threshold speeds are user definable.
- *Queue delay*, which reflects the amount of time a vehicle spends in a queued state. The properties of the trajectory that define a queued state in different tools include speed, acceleration, spacing, and number of vehicles sharing these properties. For trajectory analysis purposes, the queued state was defined previously in this chapter, and this definition is reflected in Exhibit 7-8.

For simulation tools that report total segment delay but do not report control delay explicitly, it is possible to produce approximate estimates of control delay by performing simulation runs with and without the control device(s) in place. The segment delay reported with no control is the delay due to geometrics and interaction between vehicles. The additional delay reported in the run with the control in place is, by definition, the control delay. For short segments with low to medium volumes, the segment delay usually serves as an approximation of the control delay.

The development of control delay estimates by a multiple-run procedure is primarily of academic interest because of the amount of effort involved. The objective at this point is to develop a specification for estimating control delay from vehicle trajectories that may be internalized by simulation model developers to produce HCM-compatible results.

Computational Procedures for Delay-Related Measures

The procedures for computing delay from vehicle trajectories involve aggregating all delay measures over each time step. Therefore, the results take the form of aggregated delay and not unit delay, as defined earlier. To determine unit delays, the aggregated delays must be divided by the number of vehicles involved in the aggregation. Partial trips made over a segment during the time period add some complexity to unit delay computations

The following procedures should be used to compute delay-related measures from vehicle trajectories:

- *Time step delay*: The delay on any time step is, by definition, the length of the time step minus the time it would have taken the vehicle to cover the distance traveled in the step at the target speed. This value is easily determined and is the basis for the remainder of the delay computations.

- *Segment delay*: Segment delay is represented by the time taken to traverse a segment minus the time it would have taken to traverse the segment at the target speed. The segment delay on any step is equal to the time step delay. Segment delays accumulated over all time steps in which a vehicle is present on the segment represent the segment delay for that vehicle.
- *Queue delay*: The queue delay is equal to the time step delay on any step in which the vehicle is in a queued state; otherwise, it is zero. Queue delays are accumulated over all time steps while the vehicle is in a queue.
- *Stopped delay*: The stopped delay is equal to the time step delay on any step in which the vehicle is in a stopped state; otherwise, it is zero. Since a vehicle is considered to be stopped if it is traveling at less than a threshold speed, a consistent definition of stopped delay requires that the travel time at the target speed be subtracted. Time step delays accumulated over all time steps in which the vehicle was in the stopped state represent the stopped delay. Earlier versions of this manual defined stopped delay as 76% of the control delay, on the basis of empirical data.

Queue delay computed from trajectory analysis provides the most appropriate representation of control delay.

- *Control delay*: Control delay is the additional travel time caused by operation of a traffic control device. The queue delay computed from vehicle trajectories provides a reasonable approximation of control delay when the following conditions are met:
  1. Queue delay is caused by a traffic control device, and
  2. Identification of the queued state is consistent with the definitions provided in the HCM.

Special Delay Estimation Issues

It is not possible to compute control delay from individual vehicle trajectory analysis in a manner consistent with the procedures in this manual that report control delay. It was demonstrated earlier in this chapter (see Exhibit 7-6) that the uniform delay term d_1 described in Chapter 18 is derived from trajectory analysis. The problem is that the delay adjustment terms d_2 and d_3 are macroscopic corrections that have been derived analytically. As such, they cannot be represented by vehicle trajectories. When demand volumes approach and exceed capacity, the correction terms become very large.

Exhibit 7-8 showed the trajectory of a single vehicle in an undersaturated situation. From this figure, it is observed that the control delay will be the same as the queue delay when their travel times projected to the stop line at the running speed (i.e., the broken lines) follow the same path. The problem is that the additional delays from the d_2 and d_3 adjustment terms are not represented in the figure. The adjustment terms are represented implicitly in the queue delays produced by trajectory analysis. As such, they remain a valid estimator of control delay at all levels of saturation.

While the queue delay from trajectory analysis generally provides a reasonable estimate of the delay on a controlled link, it is important to recognize certain phenomena that raise interpretation issues. The first is geometric delay, which is not included in the Chapter 18 procedure. For example, a large truck

turning right can cause additional delay to vehicles in a queue behind it. The additional delay, which would be ignored by the Chapter 18 control delay calculations, would be interpreted by trajectory analysis as control delay. This situation would cause problems in comparing the control delay estimates from the two methods.

Another problem arises with oversaturated conditions. The conceptual differences between the analytical delay model of Chapter 18 and the microscopic simulation approach make it difficult to compare their results. The comparison becomes even more complicated when queues extend into upstream links.

Density-Related Measures

Density is one of the easiest measures to compute from vehicle trajectories because it involves simply counting the vehicles in a section of roadway at a specific time. Density is therefore a product of spatial analysis as opposed to longitudinal analysis. The question is how to apply the proper definition of density to the right section of roadway over the right period of time. For example, a main obstacle in comparing densities reported by the procedural chapters in this manual with those reported by simulation tools is their different definitions. The procedures in this manual report density in terms of passenger cars per mile. Simulation tools report this measure in terms of actual vehicles per mile. The simulated densities must be converted to passenger cars per mile to produce comparable results. Procedures for conversion are discussed in Chapter 24, Concepts: Supplemental.

Because of the importance of density as a determinant of LOS, it is desirable to establish trajectory analysis procedures that are compatible with the HCM so that simulated densities can be used for LOS estimation. Microscopic simulation models establish the position of all vehicles in the system at all points in time, making it easy to define and compute density measures that are uniform among different tools by simply counting the number of vehicles on a specified portion of a roadway.

Computational Procedures

The equivalent density at a point can be determined by simulation by using a simple equation that relates density to the spacing of vehicles:

$$\text{Density (veh/mi)} = \frac{5{,}280 \text{ ft/mi}}{\text{Vehicle spacing (ft/veh)}}$$

Equation 7-1

Density can also be computed macroscopically at the segment level by simply counting the number of vehicles present on the segment during a particular time step. The densities by time step may be aggregated over an analysis period by computing the arithmetic mean of the time step densities. This method of measurement and aggregation should produce density values that are compatible with the HCM in both definition and computation, provided that the demand (d) does not exceed the capacity (c). For d/c ratios greater than 1.0, the density at the end of the analysis period may be of more interest than the average density.

Density is computed on a per lane basis in the examples given in Chapter 24. The combined density for the ramp influence area (the two freeway lanes adjacent to the ramp plus auxiliary lanes, if any, within 1,500 ft of the ramp junction) is also computed because of its application to freeway merge and diverge ramp junctions. To compute the average density for a series of segments in a freeway facility, the procedure outlined in Chapter 10 should be used.

Follower Density

This measure is defined in terms of the number of followers per mile on a two-lane highway. Follower density is not reported in this manual. Instead, percent time spent following is used as a determinant of LOS for two-lane highways in Chapter 15. The definition of the following state is given in Chapter 15 as a condition in which a vehicle is following its leader by no more than 3 s. The concept of follower density has attracted increasing international interest. It is a measure that could be easily derived from trajectory analysis.

STOCHASTIC ASPECTS OF SIMULATION ANALYSIS

The HCM's deterministic procedures give a unique result for a given set of inputs, while stochastic tools may give a distribution of results for a given set of inputs over a series of runs.

The deterministic procedures in the HCM give a unique value for all performance measures based on the specifics of the input data. Stochastic analysis tools apply a randomization process that might give different values for performance measures each time the process is repeated. In other words, simulation tools produce a distribution of values for each performance measure, much as would be expected from a series of repeated field studies. It is essential in supporting decision making to represent the distribution of values in terms of a single representative value.

A comprehensive tutorial on the stochastic aspects of simulation is presented elsewhere (*14*). Topics covered include confidence intervals, the number of runs required to achieve a specified level of confidence, and hypothesis testing for comparing alternative configurations and strategies. The tutorial material is not repeated here, but it should be understood by analysts who are using simulation to produce performance measures that are comparable with those described in this manual.

Simulation modeling is based on internally generated random numbers that are controlled by specifying an initial random number or "seed" to start the generation process. In some cases, multiple seeds are used to control different aspects of the randomization. For example, driver characteristics and vehicle characteristics might be seeded differently. Multiple runs using a simulation tool with the same input data and random number seed(s) will produce the same answers. To establish a range of answers, repetitions must be created by running a simulation tool with the same input data with different random number seed(s). Most simulation tools provide guidance on selecting random number seeds.

Number of Required Repetitions

The result of a set of simulation runs is normally represented by a summary of the average values of the performance measures of interest. Confidence in the results is influenced by the number of runs included in the set. The question

raised here is: "How many runs are needed?" The answer depends on three parameters:

1. The maximum error that can be tolerated in the results: The tolerable error may be expressed in terms of an absolute value (e.g., 5 s of delay) or as a percentage of deviation from the true mean value. Greater acceptable maximum error (tolerance) suggests the need for fewer runs.
2. The degree of confidence that the true mean falls within the specified error limits: A greater degree of confidence (i.e., 99% as opposed to 95%) suggests a need for more runs.
3. The variability across simulation runs given by the standard deviation: A greater variability (higher standard deviation) suggests a need for more runs, if the other two parameters stay fixed.

In accordance with a basic statistical approach, the standard error of the mean may be estimated from the simple relationship in Equation 7-2:

Equation 7-2

$$E = \frac{s}{\sqrt{n}}$$

where

E = standard error of the mean,

s = standard deviation of the set of runs for a particular performance measure, and

n = number of runs included in the set.

The confidence limits are expressed in terms of the number of standard errors from the mean value. A target of 95% confidence is often used for this purpose. The 95% confidence interval is represented by the mean value ±1.96 standard errors.

From the above relationship, it is possible, given the sample standard deviation s, to calculate the required sample size to produce 95% confidence of achieving a maximum tolerable error E_T by using Equation 7-3:

Equation 7-3

$$n = (1.96s)^2 / (E_T)^2$$

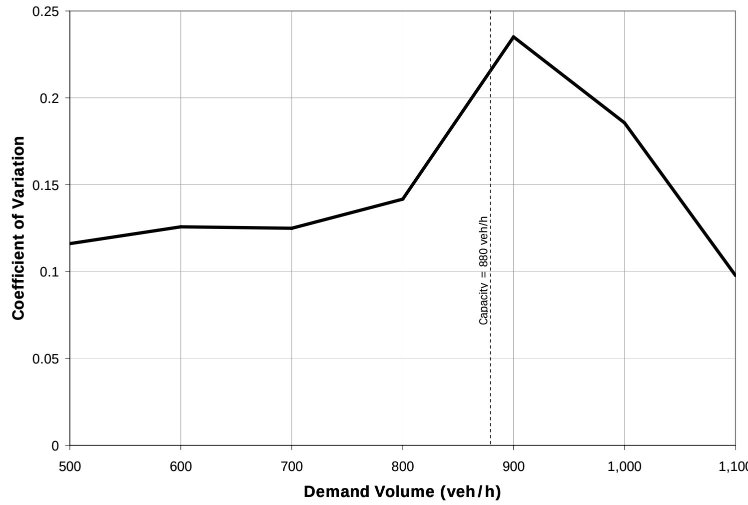
A few statistically oriented sites on the Internet offer online calculators for determining required sample sizes.

Expected Variation Between Runs

It is difficult to anticipate the amount of variation that will result from a set of runs given the input data. The standard deviation of a given performance measure is best determined by making a set of test runs and applying the sample size calculations. One factor that influences the variability at signalized intersections is the degree of saturation on each approach. This influence is illustrated in Exhibit 7-9, which shows the coefficient of variation (standard deviation/mean) on a simple signalized approach as a function of the approach volume. The data for this example included 30 runs for a 15-min period.

Exhibit 7-9
Effect of Demand Volume on Variability of Simulated Delay on an Approach to a Signalized Intersection

LIVE GRAPH
Click here to view



At low volumes, the variability is low, with the standard deviations approaching 10% of the mean value. The variability peaks at the capacity of the approach at a value near 25%. The variability is highest at capacity because some runs will see more undersaturated cycles in the operation, while others will see more oversaturated cycles. As demand volume increases well beyond approach capacity, the variability decreases significantly as deterministic phenomena begin to govern the operation.

Exhibit 7-9 shows the relationship for a single approach to an intersection. Variability may also be expected to decrease in larger systems, as illustrated in Exhibit 7-10. This example shows a very large system with 472 links, obtained from the sample data distributed with one simulation tool. The data set included 20 runs covering a 15-min period. The performance measures cover the entire system, and the resulting variation is substantially lower than would be expected on a single approach.

Exhibit 7-10
Variability of Overall Performance Measures for a Large Urban Network

Statistic	Vehicle Miles Traveled	Vehicle Hours		Minutes per Mile		Average Speed (mi/h)
		Delay	Total	Delay	Total	
Mean	19,467	238	761	0.734	2.347	25.571
Standard deviation	140	7	9	0.019	0.021	0.218
CV	0.007	0.028	0.012	0.026	0.009	0.009
Standard error	31	1.49	1.96	0.00	0.00	0.05
Upper 95%	19,528	240.497	765.197	0.742	2.356	25.667
Lower 95%	19,406	234.661	757.508	0.725	2.337	25.475

Note: CV = coefficient of variation.

COMPARING HCM ANALYSIS RESULTS WITH ALTERNATIVE TOOLS

At the start, it must be recognized that alternative traffic analysis tools have been used for many years and that not all their applications have a strong requirement for HCM compatibility. The guidance presented in this chapter and in the procedural chapters of Volumes 2 and 3 is addressed specifically to analysts who are seeking some degree of compatibility with the HCM procedures through the use of alternative tools. It is not the intent of the HCM to duplicate the tutorials and other authoritative documents in the literature dealing with the general application of traffic analysis tools.

Full numerical compatibility between the HCM and alternative tools is seldom attainable because of differences in definitions, modeling approaches, and computational methodologies.

Full numerical compatibility between the HCM and simulation-based analyses is seldom attainable because of differences in definitions, modeling approaches, and computational methodologies. An earlier section of this chapter dealt with the use of vehicle trajectory analysis to promote consistent definitions and computational procedures for the most important performance measures. The guidance in this section covers the following areas:

- Recognizing situations in which alternative tools should be applied,
- Recognizing situations in which basic incompatibilities preclude direct comparisons between the HCM and simulation results, and
- Achieving maximum compatibility between the HCM procedures and those of alternative tools.

Conceptual Differences Between Modeling Approaches

The analysis procedures described in this manual are based on deterministic models that are well founded in theory. They are implemented in the form of equations that describe the behavior of traffic. Most of the equations include empirical calibration factors derived from research. Simulation modeling, on the other hand, is based on the propagation of fictitious vehicles along a roadway segment in accordance with principles of physics, rules of the road, and driver behavior. While both modeling approaches attempt to replicate phenomena that can be observed and quantified in the field, it is sometimes difficult to obtain results that are mutually comparable. The conceptual differences that preclude comparison are discussed in the procedural chapters. A summary of key differences is presented here:

- Delays reported by the HCM interrupted-flow analysis procedures apply to all the vehicles that arrive during the analysis period. When demand volumes exceed capacity, the delay to vehicles entering the system during a given period and leaving during a subsequent period are included. Delays reported by simulation are those experienced within the analysis period regardless of when vehicles entered or left the system. This concept is explored in more detail later in this chapter in the discussion of multiperiod operation.
- Densities are reported by the uninterrupted-flow chapters in terms of passenger cars per mile. Passenger car equivalency (PCE) factors are used to convert heavy vehicles to passenger cars. PCEs are applied before the density computations. Densities reported by simulation are generally expressed in actual vehicles per mile. The effect of heavy vehicles is an

implicit result of their different characteristics. Because of this difference, it is difficult to apply PCE factors in reverse to the results of the computations.

- The procedures in this manual deal with peak 15-min-period demand flow rates, sometimes determined by applying a PHF to hourly volumes. Simulation models do not normally apply a PHF to input volumes. Therefore, some care must be taken to ensure that the demand and time periods are represented appropriately to promote the development of comparable results.
- The procedures for analysis of urban street operations focus on performance measures for arterial through vehicles. Simulation tools generally consider all vehicles, including turning movements on a street segment. To obtain comparable results from simulation, it is necessary to isolate the through movements.
- The procedures for analysis of ramp merge and diverge segments focus on traffic density within the influence of the merge area (usually the ramp and the two adjacent lanes). To obtain comparable results from simulation, it is necessary to define the merge area as a separate segment for analysis and to isolate the movements in the adjacent lanes.
- HCM procedures typically do not consider the effect of self-aggravating phenomena on the performance of a segment. For example, when traffic in a left-turn bay spills over into the adjacent through lane, the effect of the through lane performance is not considered. The inability of drivers to access their desired lane when queues back up from a downstream facility is not taken into consideration.
- Random arrivals in the traffic stream are also treated differently by the two modeling approaches. The HCM's interrupted-flow procedures apply analytical correction factors to account for this effect, while simulation modeling treats randomness explicitly by generating vehicle arrivals from statistical distributions. The difference between the two treatments affects the comparability of results.
- Some simulation tools either require or have the option of entering the origin–destination matrix instead of link and turning movement volumes. In these cases, the link and turning movement volumes are outputs from the dynamic traffic assignment models implemented as parts of the tools. HCM procedures require the link or turning movement counts as inputs.

Framework for Comparison of Performance Measures

The application framework for alternative tools is presented in the form of a flowchart in Exhibit 7-11. This framework applies to all the procedural chapters in Volumes 2 and 3.

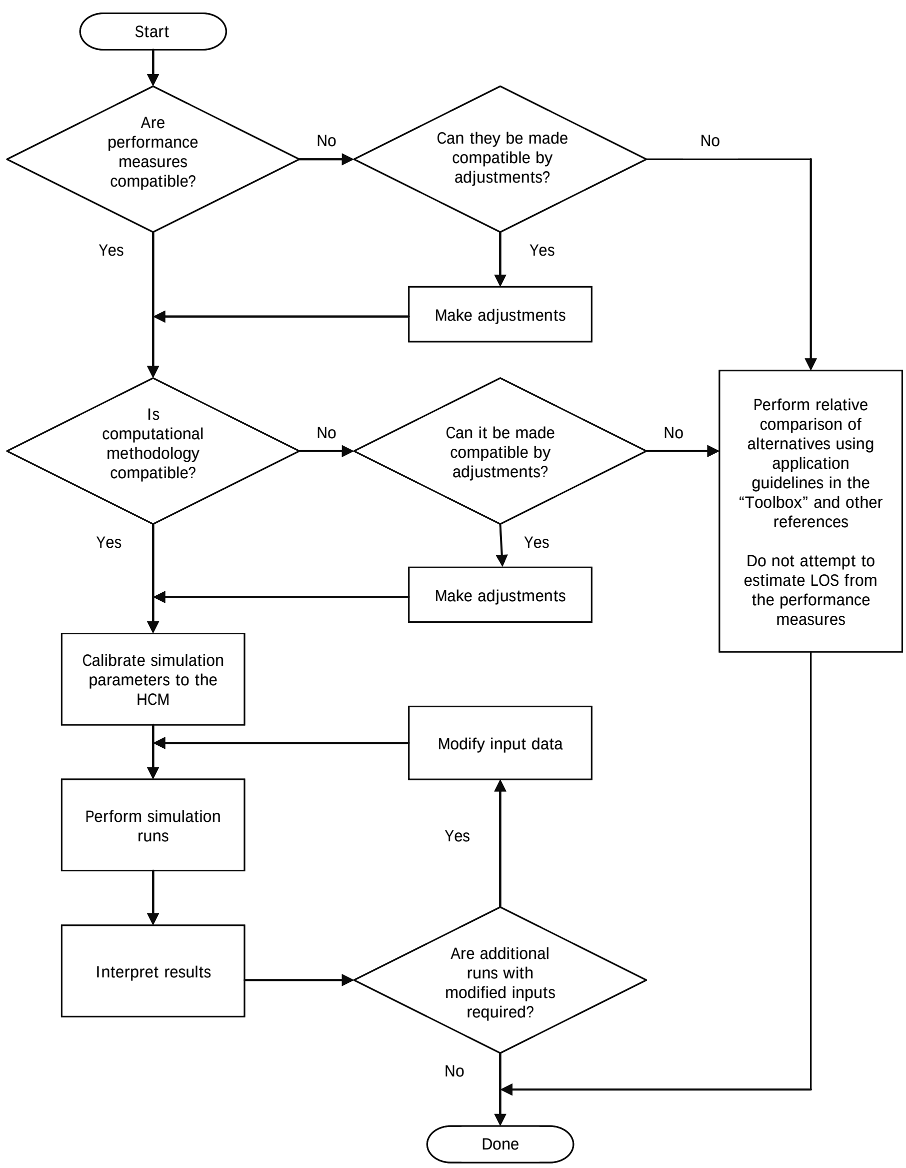
The first steps in this flowchart deal with identifying whether the situation will support analyses in which some degree of compatibility between the HCM and alternative tools may be achieved. If it is determined that, because of conceptual differences in definitions and modeling, no potential for compatibility

exists, then the use of alternative tools should be limited to feasibility assessment and comparison of candidate solutions. It is anticipated that, in most cases, there will be potential areas of compatibility.

The next steps cover the conduct of simulation analyses to achieve the desired level of compatibility with the HCM. Four steps are involved:

1. Calibrate the simulation parameters to the HCM, usually by seeking equal capacities from both processes.
2. Perform a statistically appropriate number of simulation runs.
3. Interpret the results.
4. Make iterative adjustments to calibration parameters to reconcile differences.

Exhibit 7-11
Application Framework for Alternative Tools



Alternative tools that report a performance measure with the same name as an HCM service measure, but with a different method of computation, should not be used to estimate LOS for purposes of the HCM.

LOS Comparisons

LOS estimates are determined by applying thresholds to specified performance measures (i.e., service measures). When LOS is estimated from performance measures obtained from an alternative tool, it is essential that the performance measure be determined in the same way the HCM determines the same measure. Alternative methods may be used to estimate and compare performance measures, as long as they are both trying to estimate the same fundamental measurement. Alternative tools that report a performance measure with the same name as an HCM measure, but with a different method of computation, should not be used to estimate the LOS for purposes of the HCM.

At this point, simulation tools do not generally report performance measures by using the definitions and trajectory-based method of estimation suggested in this chapter and in Chapter 24. Some refinement in the alternative tool definitions and methods of estimation based on vehicle trajectory analysis is required before valid comparisons can be made. The value of simulation modeling as a useful decision support tool is recognized, but the validity of direct comparison with performance measures defined by this manual is questionable unless the definitions and computational procedures conform to those prescribed in this chapter.

HCM LOS thresholds are often based on service measures representing the peak 15 min of demand (arriving vehicles) rather than the 15-min period when the measure reached its maximum value.

It is also important to note that the HCM applies LOS thresholds to performance measures that represent the peak 15 min of demand (i.e., arriving vehicles) and not necessarily the 15-min period when the performance measure produced its maximum value.

The presence of significant queues at the end of an analysis period can often be taken as an indicator that LOS F has been reached.

One consideration that makes simulation more compatible with the HCM in reporting LOS is the criterion that, for most roadway segments, LOS F is assigned to any segment that operates above its capacity. Therefore, without the need for a detailed trajectory analysis, the presence of significant queues at the end of the analysis period can be taken as an indicator that LOS F has been reached in the segment. When queues extend into a given segment from a downstream bottleneck, it is important to use the analysis procedures for freeway facilities described in Chapter 10 of this manual instead of the procedures for individual segments described in Chapters 11 to 13. On the other hand, when the purpose of the analysis is to develop a facility design that will produce a LOS better than F, the analyst must ensure that the performance measure on which LOS is based is estimated in a manner compatible with the HCM.

Estimation of Capacity by Simulation

It is commonly accepted that the capacity of an approach or segment may be estimated by overloading it and observing the maximum throughput. This technique is valid in some cases, but it must be used with caution when congestion could become a self-aggravating phenomenon. For example, when lane selection is important (as in the case of a turning bay) and congestion keeps vehicles from their desired lane, the throughput can drop below its theoretical maximum. This phenomenon is not recognized by most of the HCM's deterministic analysis procedures. Therefore, if the objective is to seek HCM-compatible capacity levels, the approach or segment should not be overloaded

by more than a few percent. In this case, the process of determining capacity might require iteration. On the other hand, if the objective is to evaluate the operation under an anticipated heavy overload, then simulation modeling might provide some insight into the nature of the resulting congestion. In that case, the analysis could require development of the relationship between demand and throughput. Examples of the adverse effects of heavy overloading are presented in Chapter 27, Freeway Weaving: Supplemental, and Chapter 34, Interchange Ramp Terminals: Supplemental.

Temporal and Spatial Boundaries

The LOS reported by the HCM procedures applies to the 15-min period with the maximum number of arrivals (i.e., entering vehicles). This period might not be the same one that reports the maximum delay because of residual queues. In a discussion of the limitations of performance measure estimation and use (*15*), there is frequent reference to the issues that arise in the treatment of incomplete trips within the analysis period, including those that entered the special domain of the analysis but did not exit during the analysis period and those that were unable to enter the spatial domain because of queue backup. The main problem lies in differences in treatment among different models.

Complete Versus Incomplete Trips

Definition of incomplete trips within the temporal and spatial boundaries of an analysis.

Five categories are proposed with respect to incomplete trips (*15*):

1. Vehicles that were present at the start of the analysis period and were able to exit the system successfully before the end of the analysis period;
2. Vehicles that were present at the start of the analysis period but were unable to exit the system successfully before the end of the analysis period;
3. Vehicles that were able to enter the system during the analysis period but were unable to exit the system successfully before the end of the analysis period;
4. Vehicles that tried to enter the system during the analysis period but were unsuccessful; and
5. Vehicles that entered during the analysis period and were able to exit the system successfully before the end of the analysis period.

All categories except the fifth represent incomplete trips. It is suggested elsewhere (*15*) that, if a specific analysis contains more than 5% incomplete trips, the period length should be increased.

It is important to recognize that the objectives of the Federal Highway Administration's *Traffic Analysis Toolbox* (*16*) differ somewhat from those of the HCM. The purpose of the *Toolbox* is to provide general guidance on applying traffic analysis tools. The guidance on simulation included in this chapter is more focused on developing HCM-compatible performance measures so that those measures can be used in conjunction with the HCM procedures. Therefore, this discussion must examine temporal and spatial boundaries from the same perspective as the HCM procedures.

When undersaturated operation is being studied, the definition of the facility in time and space is much less important. The operation tends to be more homogeneous when *d/c* ratios are less than 1.00. Extending the analysis period will give a larger sample of vehicles for most performance measures but will not affect the measures significantly.

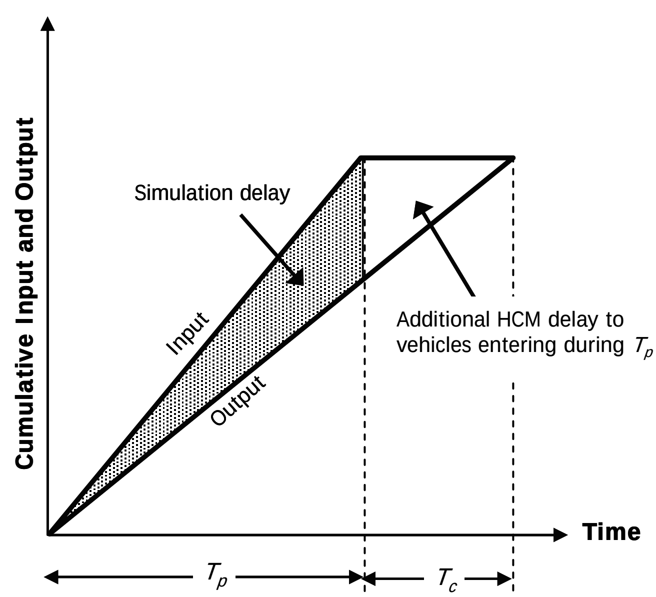
The issues are more conspicuous when the *d/c* ratio is greater than 1.00 for short periods. In this case, queues build up and the analysis (either HCM or simulation) must define temporal boundaries that begin and end without congestion. It is also desirable, but not essential, that the spatial boundaries encompass uncongested operation. Failure to define a spatially adequate system will result in vehicles being denied entry, but these vehicles will eventually be processed if the analysis period is long enough.

Delay on Oversaturated Signalized Approaches

LOS for interrupted flow is defined by the HCM in terms of the delay to all vehicles *entering* the facility during the analysis period. All vehicles wishing to enter are assumed to enter. Those unable to exit from a signalized intersection are accumulated in a residual queue and are assumed to exit later. The incremental (d_2) term of the delay model accounts for delay to vehicles that exit in a later period. The d_3 term accounts for the additional delay caused by an initial queue.

The formulation illustrated in Exhibit 7-12 recognizes that delay accrues when the vehicular input to a system exceeds the output for a period of time. The HCM uses this formulation to estimate delay that accrues at a signalized intersection when volume exceeds capacity over the analysis time period, T_p. The HCM delay in Exhibit 7-12 is represented by the area of the two triangles shown in the figure. The area within the two triangles is referred to as the *deterministic queue delay* (DQD). The DQD may be determined as $5 \times T_p \times (X - 1)$, where X is the *d/c* ratio.

Exhibit 7-12
Oversaturated Delay Representation by the HCM and Simulation Modeling



When demand exceeds capacity, some vehicles that arrive during T_p will depart during the next period. The time required to clear all vehicles arriving during T_p is shown above as T_c. Because the HCM defines delay in terms of the delay experienced by *all vehicles that arrive* during the analysis period, the delay computations must include the delay to those vehicles that arrive during T_p and depart during T_c.

The HCM defines delay in terms of the delay experienced by all vehicles arriving during an analysis period (e.g., 15 min), including delay accumulated after the end of the analysis period.

This definition differs from the delay definition used by most simulation tools, which address the delay experienced *during* the analysis period. The HCM definition includes the area within both triangles of Exhibit 7-12. The simulation definition includes only that portion of the area within the interval T_p.

Most simulation tools define delay in terms of the delay experienced by all vehicles during a specified analysis period and do not include delay from later time periods.

Compatibility with the HCM definition dictates that a control delay measure should be based on all entering vehicles, without regard to completed trips. An adequate initialization period should be used to load the facility. When the *d/c* ratio is less than 1.00, some vehicles that entered before the start of the analysis (i.e., during the initialization period) will exit the system. There will also be vehicles that enter the system late in the period and do not exit. Including these incomplete trips will not bias the delay results.

When demand exceeds capacity for a single period, the HCM delay formulation shown in Exhibit 7-12 will include the delay to vehicles that exit in the next period. The simulation results will not. To produce a simulation run that replicates the HCM single-period calculations, a second period with zero demand must be added to the simulation run. Only the vehicles that were unable to exit during the first period will be accommodated during the second period. The sum of the delays for both periods will be equivalent to the HCM delay shown in Exhibit 7-12.

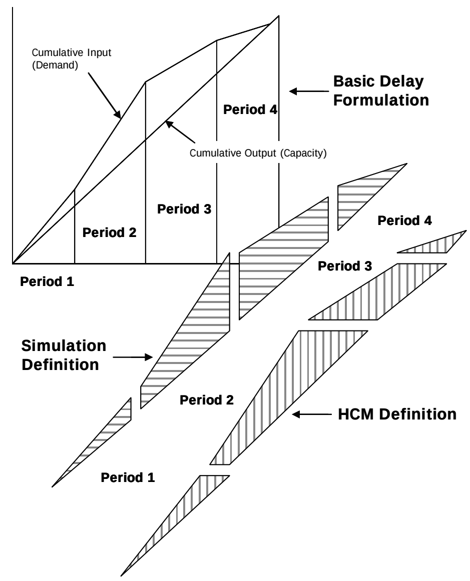
Delay for Multiperiod Oversaturation

When the operation is oversaturated beyond a single period, it is necessary to perform a multiperiod analysis, ensuring that the duration is sufficient to encompass congestion-free conditions at both ends.

When operations are oversaturated beyond a single analysis period, a multiperiod analysis is necessary.

As an example, HCM and simulation delay formulations are illustrated in Exhibit 7-13, which depicts the analysis of four consecutive periods that begin and end without congestion. The analysis is performed sequentially, with the residual queues from one period applied as initial queues to the next period. The first two periods have demand in excess of capacity. In the last two periods, the demand drops sufficiently below capacity to allow the queues to clear. Delay polygons are shown for the HCM and simulation definitions for all periods. The shape of the delay polygons differs in the two formulations, so the delay values are not the same for any period. The important thing is that the sum of the areas for the four polygons is the same for each definition.

Exhibit 7-13
Comparison of HCM and Simulation Delay Definitions for Four Oversaturated Periods



Therefore, to promote compatibility between the HCM and simulation delay definitions for a multiperiod analysis involving oversaturated signalized approaches, the simulation results should be obtained as follows:

- Ensure that the analysis period is long enough to encompass a period of congestion-free operation at both ends.
- Perform an adequate initialization to load the system.
- Perform the analysis on all vehicles entering the system during each period.
- Do not ignore any entering or exiting vehicle in any period; otherwise, the results could be biased.
- If a measure of delay per vehicle is desired, develop the total delay by summing the delays for the individual periods and divide that delay by the total entering volume.

Delay is not reported explicitly in the freeway segment chapters (Chapters 11 to 13). However, delay may be inferred from each chapter's free-flow and average speed computations. This step is performed in Chapter 10 for analysis of freeway facilities involving a combination of different segment types. The delay due to queues forming from bottlenecks is added to the individual segment delays. While the delay computations are conceptually simpler for freeways, the same guidance for developing compatible simulation results applies to other system elements.

Density is defined only in the uninterrupted-flow chapters. Unlike delay measures, which apply to individual vehicles, the density measure applies to the facility. Therefore, the issue of how to treat incomplete trips does not apply. Instantaneous densities should be determined from simulation by time step and should be aggregated over suitable intervals. The average density over a long period will be of less interest for most purposes than the variation of density that takes place in time and space. Typical aggregation intervals for that purpose will range from 5 to 15 min.

4. PRESENTATION OF RESULTS

GUIDANCE ON THE DISPLAY OF HCM RESULTS

Conventions for the display of results in the HCM.

Tabular values and calculated results are displayed in a consistent manner throughout the HCM. It is suggested that analysts adhere to these conventions. A key objective is to use the number of significant digits that is reasonable, to indicate to users, decision makers, and other viewers that the results are not extremely precise but take on the precision and accuracy associated with the input variables used. This guidance applies primarily to inputs and final outputs; intermediate results in a series of calculations should not be rounded unless specifically indicated by a particular methodology.

Input Values

Following is a list of representative (not exhaustive) input variables and the suggested number of digits for each.

- Volume (whole number);
- Grade (whole number);
- Lane width (one decimal place);
- Percentage of heavy vehicles (whole number);
- PHF (two decimal places);
- Pedestrian volume (whole number);
- Bicycle volume (whole number);
- Parking maneuvers (whole number);
- Bus stopping (whole number);
- Green, yellow, all-red, and cycle times (one decimal place);
- Lost time/phase (whole number); and
- Minimum pedestrian time (one decimal place).

Adjustment Factors

Factors interpolated from tabular material can use one more decimal place than is presented in the table. Factors generated from equations can be taken to three decimal places.

Service Volume Tables

When rounding volumes for service volume tables, a precision no greater than the nearest 10 vehicles or passenger cars should be used for hourly tables and the nearest 100 vehicles or passenger cars for daily tables.

Free-Flow Speed

For base free-flow speeds, show the value to the nearest 1 mi/h. For free-flow speeds that have been adjusted for various conditions and that are considered intermediate calculations, show speed to the nearest 0.1 mi/h.

Speeds

For threshold values that define LOS, show speed to the nearest 1 mi/h. For intermediate calculations of speed, use one decimal place.

Volume-to-Capacity and Demand-to-Capacity Ratios

Show v/c and d/c ratios with two decimal places.

Delay

In computing delay, show results with one decimal place. In presenting delay as a threshold value in LOS tables, show a whole number.

Density

Show density results with one decimal place.

Pedestrian Space

Show pedestrian space values with one decimal place.

Occurrences and Events

For all event-based items, use values to a whole number. These items include parking maneuvers, bus stops, events along a pedestrian or bicycle path, and number of cycles in a given time period.

General Factors

In performing all calculations on a computer, the full precision available should be used. Intermediate calculation outputs should be displayed to three significant digits throughout. For the measure that defines LOS, the number of significant digits presented should exceed by one the number of significant digits shown in the LOS table.

PRESENTING RESULTS TO FACILITATE INTERPRETATION

Performance measures selected should be related to the problem being addressed.

Several performance measures can result from HCM analyses. Determining the most appropriate measures to use for a decision depends on the particular case. However, decision-making situations generally can be divided into those involving the public (e.g., city councils and community groups) and those involving technicians (e.g., state and local engineering and planning staff).

The HCM is highly technical and complex. The results of the analyses can be difficult for people to interpret for decision making, unless the data are carefully organized and presented. In general, the results should be presented as simply as possible. This presentation might use a small set of performance measures and provide the data in an aggregate form without losing the ability to relate to the underlying variations and factors that generated the results.

The LOS concept was created, in part, to make presentation of results easier than if numerical values of service measures were reported directly. In many cases, analysts and decision makers prefer to see one service measure rather than multiple performance measures. At the same time, relying solely on LOS results in making recommendations or decisions can result in important information available from other performance measures being overlooked. Nevertheless,

although there are limitations to its usefulness, the LOS concept remains a part of the HCM because of its acceptance by the public and decision makers.

Decision makers who represent the public usually prefer measures that their constituents can understand; the public can relate to LOS results. Unit delay (e.g., seconds per vehicle) and travel speed are also readily understood. However, volume-to-capacity ratio, density, percent time spent following, and vehicle hours of travel are not measures to which the public easily relates. When the measures to present are selected, it is important for the analyst to recognize the orientation of the decision maker and the context in which the decision will be made. In general, these measures can be differentiated as system-user or system-manager oriented.

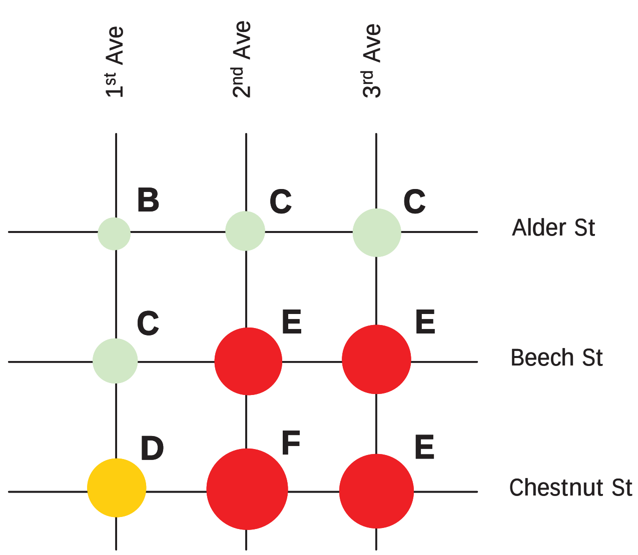
GRAPHIC REPRESENTATION OF RESULTS

Historically, data and analysis results have been presented primarily in tables. However, results may be best presented as pictures and supplemented only as necessary with the underlying numbers in some situations. Graphs and charts should not be used to decorate data or to make dull data entertaining; they should be conceived and fashioned to aid in interpretation of the meaning behind the numbers (*17*).

Present results to make them very plain (obvious) to the audience.

Most performance measures in the HCM are quantitative, continuous variables. LOS values, however, result from step functions and do not lend themselves to graphing. When placed on a scale, LOS results must be given an equivalent numeric value, as shown in Exhibit 7-14, which presents the LOS for a group of intersections. The LOS letter is indicated, and shaded (or colored) areas indicate intersections that are below, at, or above the analysis objective of LOS D. The size of the indicator at each intersection shows the relative control delay value for the indicated LOS.

Exhibit 7-14
Example of a Graphic Display of LOS



The issue is whether the change in value between successive LOS values (i.e., the interval) should be equal. For instance, is it appropriate for LOS A to F to be converted to a scale of 0 through 5? Should the numerical equivalent assigned to the difference of the thresholds between LOS A and B be the same as the difference between LOS E and F? These questions have not been addressed in research, except in the area of traveler-perception models. Furthermore, LOS F is not given an upper bound. Therefore, a graph of LOS should be considered ordinal, not interval, because the numeric differences between the levels would not appear significant.

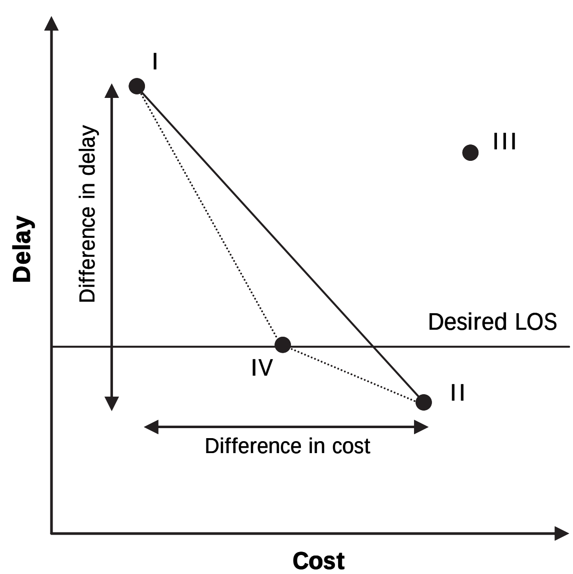
However, it is difficult to refrain from comparing the differences. A scale representing the relative values of the LOS letters would have to incorporate the judgment of the analyst and the opinions of the public or decision makers—a difficult task. A thematic graphic presentation, however, avoids this issue. In Exhibit 7-15, for example, shading is used to highlight time periods and basic freeway segments that do not meet the objective LOS (in this case, D).

Exhibit 7-15
Example of a Thematic Graphic Display of LOS

Start Time	Segment 1	Segment 2	Segment 3	Segment 4
5:00 p.m.	A	B	B	A
5:15 p.m.	B	B	D	A
5:30 p.m.	B	B	F	A
5:45 p.m.	B	D	F	A
6:00 p.m.	B	F	F	A
6:15 p.m.	D	F	E	A
6:30 p.m.	D	E	C	A
6:45 p.m.	B	B	B	A

Simple graphics often can facilitate decision making among available alternatives. For example, in the cost-effectiveness graph in Exhibit 7-16, the estimated delays resulting from alternative treatments are plotted against their associated cost. The graph shows more clearly than a tabulation of the numbers that Alternative III is more costly and creates higher delay than Alternative II. This result eliminates Alternative III.

Exhibit 7-16
Example of a Cost-Effectiveness Graph



Whether Alternative I or II should be chosen, however, is a matter for the decision maker's judgment. Alternative II is more expensive than Alternative I but is predicted to deliver a significantly lower delay. A useful measure for decision makers is provided by the slope of the line between the alternatives, which shows the seconds of delay saved per dollar of cost.

For this example, assume that Alternative IV provides the minimum acceptable LOS at significantly less cost than Alternative III. The dotted lines in Exhibit 7-16 indicate the relative cost-effectiveness of moving from Alternative I to IV or Alternative IV to II. The steepest slope—I to IV—signifies a high level of cost-effectiveness. The two alternatives that meet or exceed the LOS objective are Alternatives II and IV. The most appropriate alternative, therefore, is Alternative IV.

The HCM provides valuable assistance in making transportation management decisions in a wide range of situations. It offers the user a selection of performance measures to meet a variety of needs. The analyst should recognize that using the HCM involves mixing a bit of art along with the science. Sound judgment is needed not only for interpreting the values produced but also for summarizing and presenting the results.

5. REFERENCES

Many of these references can be found in the Technical Reference Library in Volume 4.

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Persons per hour, passenger-car equivalents per hour, and vehicles per hour are measures that can define capacity, depending on the type of roadway element and the type of analysis being conducted. The concept of person-flow is important (*a*) in making strategic decisions about transportation modes in heavily traveled corridors and (*b*) in defining the role of transit and high-occupancy-vehicle priority treatments. Person-capacity and person-flow weight each type of vehicle in the traffic stream by the number of occupants carried.

UNINTERRUPTED-FLOW ROADWAYS

Characteristics

Uninterrupted-flow roadways have no fixed causes of delay or interruptions to the traffic stream such as traffic signals. Freeways and their components operate under the purest form of uninterrupted flow. There are no fixed interruptions to traffic flow, and access is controlled and limited to ramp locations. Multilane highways and two-lane highways can also operate under uninterrupted flow in long segments; however, it may also be necessary to examine points along those highways where traffic may need to slow or stop (e.g., traffic signals, roundabouts, and STOP signs).

The traffic stream on uninterrupted-flow facilities is the result of individual vehicles interacting with each other and the facility's geometric characteristics. The pattern of flow is generally controlled only by the characteristics of the land uses that generate the traffic using the facility, although freeway management and operations strategies—such as ramp metering, freeway auxiliary lanes, truck lane restrictions, variable speed limits, and incident detection and clearance—can also influence traffic flow. Operations can also be affected by environmental conditions, such as weather or lighting, by pavement conditions, and by the occurrence of traffic incidents (*1*, *2*).

"Uninterrupted flow" describes the type of facility, not the quality of the traffic flow at any given time. A freeway experiencing stop-and-go congestion, for example, is still an uninterrupted-flow facility, despite the congestion.

HCM Methodologies

The HCM provides methodologies for the following uninterrupted-flow system elements:

- *Freeway facilities.* An extended length of a single freeway composed of a set of connected basic freeway, weaving, and merge and diverge segments.
- *Basic freeway segments.* The portions of a freeway outside the influence area of any on- or off-ramps.
- *Freeway weaving segments.* The portions of a freeway where an on-ramp is closely followed by an off-ramp and entering or exiting traffic must make at least one lane change to enter or exit the freeway.
- *Freeway merge and diverge segments.* The portions of a freeway where traffic enters or exits without having to change lanes to enter or leave a through traffic lane.

- *Multilane highways.* Higher-speed facilities, with two or more lanes in each direction, without full access control (i.e., traffic enters and exits by means of at-grade intersections, which may or may not be signal-controlled).
- *Two-lane highways.* Facilities with mostly one lane of travel per direction, with motorists using passing lanes, turnouts, or the opposing lane (where allowed by regulation and opposing traffic) to pass slower vehicles.

Performance Measures

The following are key performance measures used to evaluate the operation of uninterrupted-flow roadways:

- *Density* is typically defined by the average number of vehicles (or passenger-car equivalents) per lane mile of roadway. The denser the traffic conditions, the closer vehicles are to each other and the harder it is for vehicles to change lanes or maintain a constant speed. Density is frequently used to evaluate freeways and multilane highways.
- *Speed* reflects how fast motorists can travel along a roadway. The speed that a motorist would travel along an uninterrupted-flow roadway under low-volume conditions is known as the *free-flow speed.* Drivers experience *delay* when their travel speed is less than the free-flow speed, which is a result of traffic demands approaching or exceeding the roadway's capacity. Speed is used to evaluate all kinds of uninterrupted-flow roadways.
- *Percent time-spent-following* is a measure specific to two-lane highways. It represents the freedom to maneuver and the comfort and convenience of travel. It is the average percentage of travel time that vehicles must travel in platoons behind slower vehicles because of the inability to pass.
- *Volume-to-capacity (v/c) ratio* reflects how closely a roadway is operating to its capacity. By definition, the volume of traffic using a roadway cannot exceed the roadway's capacity. Therefore, the *v/c* ratio is actually a *demand-to-capacity (d/c) ratio*. However, *v/c* ratio is the historically used term. A *v/c* ratio that exceeds 1.00 indicates that more vehicles demand to use a roadway than can be accommodated. *Observed volumes* represent only the number of vehicles that are able to pass through a portion of roadway under existing conditions; these volumes may be constrained by bottlenecks elsewhere along the roadway. In contrast, *demand volumes* represent the number of vehicles that would be observed in the absence of any bottlenecks. Analyses that fail to account for demand volumes may result in a forecast of better future conditions than would actually occur.

INTERRUPTED-FLOW ROADWAYS

Characteristics

Interrupted-flow facilities have fixed causes of periodic delay or traffic stream interruption, such as traffic signals, roundabouts, and STOP signs. Urban streets are the most common form of this kind of facility. Exclusive pedestrian and bicycle facilities are also treated as interrupted flow, since they may

occasionally intersect other streets at locations where pedestrians and bicyclists do not automatically receive the right-of-way.

The traffic flow patterns on an interrupted-flow facility are the result of vehicle interactions, the facility's geometric characteristics, the traffic control used at intersections, and the frequency of access points to the facility. Traffic signals, for example, allow designated movements to occur only during certain portions of the signal cycle (and, therefore, only during certain portions of an hour). This control creates two significant outcomes. First, time affects flow and capacity, since the facility is not available for continuous use. Second, the traffic flow pattern is dictated by the type of control used. For instance, traffic signals create platoons of vehicles that travel along the facility as a group, with significant gaps between one platoon and the next. In contrast, all-way STOP-controlled intersections and roundabouts discharge vehicles more randomly, creating small (but not necessarily usable) gaps in traffic at downstream locations (*1*, *3*).

HCM Methodologies

The HCM provides methodologies for the following interrupted-flow system elements:

- *Urban street facilities,* which are extended sections of collector or arterial streets, and which include the impacts of traffic signals or other traffic control along the street. Urban street facilities are formed by two or more *urban street segments,* which are typically street sections extending from one traffic signal to the next, including the impact of the traffic signal at the far ("downstream") end of the segment. Roundabouts and STOP-sign control on the major street can also define the end of an urban street segment. Segments are the basic analysis unit for multimodal analyses.
- *Signalized intersections.*
- *Interchange ramp terminals,* which are two closely spaced intersections of freeway ramps and surface streets. The HCM can be used to analyze interchange ramp terminals consisting of a pair of signalized intersections or a pair of roundabouts.
- *Unsignalized intersections,* including two-way STOP-controlled intersections (i.e., intersections where only the side-street approaches are required to stop), all-way STOP-controlled intersections, and roundabouts.
- *Off-street pedestrian and bicycle facilities,* such as bicycle paths or multiuse trails. On-street pedestrian and bicycle facilities are addressed by the methodologies for urban streets and intersections, although not every system element has an associated pedestrian or bicycle methodology.

Performance Measures

The following are key performance measures used in evaluating the operation of motorized vehicles on interrupted-flow roadways:

- *Control delay* is the delay incurred because of the presence of a traffic-control device. It includes delay associated with vehicles slowing in advance of an intersection, the time spent stopped on intersection approach, the time spent as vehicles move through a queue, and the time needed for vehicles to accelerate to their desired speed once through the intersection.
- *Speed* reflects how fast motorists can traverse a roadway section, including the effects of traffic-control devices, delays due to turning vehicles at intersections and driveways, and traffic demands on the roadway.
- *Number of stops* reflects how frequently motorists must come to a stop while traveling along an urban street because of traffic control, turning vehicles, midblock pedestrian crossings, and similar factors.
- *Queue length* reflects how far traffic backs up as a result of traffic control (e.g., a queue from a traffic signal) or a vehicle stopped in the travel lane while waiting to make a turn. Queuing is both an important operational measure and a design consideration—queues that are longer than the available storage length can create several types of operational problems. A through-lane queue that extends past the entrance to a turn lane blocks access to the turn lane, keeping it from being used effectively. Similarly, a turn-lane queue overflow into a through lane interferes with the movement of through vehicles. Queues that extend upstream from an intersection can block access into and out of driveways and—in a worst case—can spill back into and block upstream intersections, causing side streets to begin to queue back.
- *Volume-to-capacity (demand-to-capacity) ratios,* which are defined and used similarly to those of uninterrupted-flow roadways.
- The performance measures produced by *traveler-perception models* describe how travelers would perceive conditions. These models use a variety of inputs to generate a single performance measure. The measure value predicts the average perception rating that all users of a given mode would give a particular system element. Traveler-perception models are frequently applied to pedestrian, bicycle, and transit analyses and are discussed further in Section 3, Quality and Level-of-Service Concepts.
- *Pedestrian space, bicycle speed,* and *number of meeting/passing events* on off-street pedestrian and bicycle facilities can also be of interest to analyses involving the pedestrian and bicycle modes.

MODAL INTERACTIONS

Roadways serve users of many different modes: in particular, motorists, pedestrians, bicyclists, and transit passengers. The roadway right-of-way is allocated among the modes through the provision of facilities that ideally serve each mode's needs. However, in many urban situations, the right-of-way is

constrained by adjacent land development, causing transportation engineers and planners to consider trade-offs in how to allocate the right-of-way. Interactions among the modes that result from different right-of-way allocations are important to consider in analyzing a roadway, and the HCM provides tools for assessing these interactions. Local policies and design standards relating to roadway functional classifications also provide guidance on the allocation of right-of-way; safety and operational concerns should also be addressed. Exhibit 8-1 summarizes some of the key interactions that occur between modes.

Exhibit 8-1
Modal Interaction Summary

Mode Creating the Interaction	Mode Affected by the Interaction			
	Automobile	**Pedestrian**	**Bicycle**	**Transit**
Automobile	Turning automobiles can delay other automobiles; heavy vehicles have poorer acceleration and deceleration characteristics; traffic signal timing is influenced by relative traffic volumes on intersection approaches; intersection delay tends to increase as automobile volumes increase	Cross-street automobile volumes influence traffic signal timing (and pedestrian delay); turning movement conflicts between cars and pedestrians; automobile and heavy vehicle volumes and their perceived separation from pedestrians using sidewalks	Automobile and heavy vehicle volumes and speeds, presence of on-street parking, and the degree to which bicyclists are separated from vehicular traffic influence bicyclist comfort; turning movement conflicts with automobiles at intersections	Impacts similar to those of automobiles on automobiles; buses may be delayed waiting for a gap in traffic when they leave a bus stop; day-to-day variations in traffic volumes and trip-to-trip variations in making or missing green lights affect schedule reliability
Pedestrian	Minimum green times at traffic signals may be dictated by crosswalk lengths; automobiles yield to crossing pedestrians	Cross-flows where pedestrian flows intersect cause pedestrians to adjust their course and speed; pedestrian space and comfort decrease as pedestrian volumes increase	Pedestrians being met and passed by bicycles on multiuse paths affect bicyclist comfort because of pedestrians' lower speeds and tendency to walk abreast; on streets, effect on bicycles similar to that on automobiles	Impacts similar to those of pedestrians on automobiles; transit riders are often pedestrians before and after their transit trip, so the quality of the pedestrian environment affects the perceived quality of the transit trip
Bicycle	Turning automobiles yield to bicycles; automobiles may be delayed waiting to pass bicycles in shared-lane situations	Bicycles meeting and passing pedestrians on multiuse paths affect pedestrian comfort because of the bicycles' markedly higher speeds	Bicyclists may be delayed when they pass another bicycle on-street; meeting and passing events on off-street pathways affect bicyclist comfort	Impacts similar to those of bicyclists on automobiles; bicycles can help extend the area served by a transit stop
Transit	Buses are heavy vehicles; buses stopping in the travel lane to serve passengers can delay other vehicles; transit signal priority measures affect the allocation of green time	Impacts similar to those of automobiles on pedestrians, but proportionately greater due to transit vehicles' greater size	Impacts similar to those of automobiles on bicyclists, but proportionately greater due to transit vehicles' greater size; transit can help extend the reach of a bicycle trip and allows a trip to be completed in the event of a flat tire or rain	Bus speeds decrease as bus volumes increase; irregular headways increase passenger loads on some buses and increase average wait times for buses

3. QUALITY AND LEVEL-OF-SERVICE CONCEPTS

OVERVIEW

There are many ways to measure the performance of a transportation facility or service—and many points of view that can be considered in making that measurement. The agency operating a roadway, automobile drivers, pedestrians, bicyclists, bus passengers, decision makers, and the community at large all have their own perspectives on how a roadway or service should perform and what constitutes "good" performance. As a result, there is no one right way to measure and interpret performance. The HCM provides a number of tools for describing how well a transportation facility or service operates from a traveler's perspective, a concept termed *quality of service.* One important tool for describing quality of service is the concept of LOS, which facilitates the presentation of results through the use of a familiar A (best) to F (worst) scale. A variety of specific performance measures, termed *service measures,* are used to determine LOS. These three concepts—quality of service, LOS, and service measures—are the topics of this section.

QUALITY OF SERVICE

Quality of service describes how well a transportation facility or service operates from a traveler's perspective.

Quality of service describes how well a transportation facility or service operates from a traveler's perspective. Quality of service can be assessed in a number of ways, including directly observing factors perceivable by and important to travelers (e.g., speed or delay), surveying travelers, tracking complaints and compliments about roadway conditions, forecasting traveler satisfaction by using models derived from past traveler surveys, and observing things not directly perceived by travelers (e.g., average time to clear a crash) that affect things they can perceive (e.g., speed or arrival time at work).

The HCM's focus is on the travel time, speed, delay, ability to maneuver, and comfort aspects of quality of service. Travel time reliability is expected to be added in the future, as a result of funded research. Other aspects of quality of service covered to a lesser degree by the HCM, or covered more thoroughly by its companion documents, include convenience of travel, safety, user cost, availability of facilities and services, roadway aesthetics, and information availability.

Dimensions of system performance and mobility.

Quality of service is one dimension of mobility and overall transportation system performance. Other dimensions to consider are the following (*4*, *5*):

- *Quantity of service*—such as the number of person miles and person-hours provided by the system;
- *Capacity utilization*—including the amount of congestion experienced by users of the system, the physical length of the congested system, and the number of hours that congestion exists; and
- *Accessibility*—for example, the percentage of the populace able to complete a selected trip within a specified time.

LEVEL OF SERVICE

LOS is the stratification of quality of service.

The HCM defines LOS for most combinations of travel mode (i.e., automobile, pedestrian, bicycle, and transit) and roadway system element (e.g., freeway, urban street, intersection) addressed by HCM methodologies. Six levels are defined, ranging from A to F. LOS A represents the best operating conditions from the traveler's perspective and LOS F the worst. For cost, environmental impact, and other reasons, roadways and transit services are not typically designed to provide LOS A conditions during peak periods. Rather, a lower LOS that reflects a balance between individual travelers' desires and society's desires and financial resources is typically the goal. Nevertheless, during low-volume periods of the day, a system element may operate at LOS A.

LOS is used to translate complex numerical performance results into a simple A–F system representative of the travelers' perceptions of the quality of service provided by a facility or service. The LOS letter result hides much of the complexity of facility performance to simplify decision making about whether facility performance is generally acceptable and whether a change in this performance is likely to be perceived as significant by the general public. One of the strengths of the LOS system, and a reason for its widespread adoption by agencies, is its ability to communicate roadway performance to laypersons. However, LOS has other strengths and weaknesses, described below, that both analysts and decision makers need to be mindful of.

Step Function Nature of LOS

Defining performance standards on the basis of LOS (or any fixed numerical value) means that small changes in performance can sometimes result in the standard being exceeded, when a facility is already operating close to the standard.

The measure of effectiveness for automobiles at traffic signals is the average delay experienced by motorists. As traffic volumes on certain critical approaches increase, so does the average delay. The added delay may or may not result in a change in LOS. An increase of delay of 12 s may result in no change in LOS, a drop of one LOS level, or a drop of two LOS levels, depending on the starting value of delay. Because there are only six possible LOS letters, each covering a range of possible values, the reported LOS does not change until the service measure increases past the threshold value for a given LOS. A change of LOS indicates that roadway performance has transitioned from one given range of traveler-perceivable conditions to another range, while no change in LOS indicates that conditions are in the same performance range as before. The service measure value—in this case, average delay—indicates more specifically where conditions lie within a particular performance range.

Because a small change in a service measure can sometimes result in a letter change in the LOS result, the LOS result may imply a more significant effect than actually occurred. This aspect of LOS can be a particularly sensitive issue when agencies define their performance standards on the basis of LOS, since a small change in performance can trigger the need for potentially costly improvements. However, this issue exists whenever a fixed standard is used, whether or not LOS is the basis of that standard.

Uncertainty and False Precision

Computer software is frequently used to perform traffic operations analyses, and software can report results to many decimal places. However, such precision is often unjustified for four reasons:

1. In contrast to the force of gravity or the flow of water through a pipe, the actions of motorists driving on a roadway can vary. Traffic operations models predict average values of performance measures; the actual value for a measure on a given day may be somewhat higher or lower. Thus, the result reported by every traffic operations model has some uncertainty associated with it.
2. A given traffic operations model may rely on the output of other models that have their own associated result uncertainties.
3. Some model inputs, such as traffic volumes, are taken to be absolute, when there is actually variation in the inputs from month to month, day to day, or even within an hour. Traffic volumes, for example, may vary by 5% to 10% from one weekday to the next.
4. Some HCM models predict traveler perceptions. Two travelers who experience identical conditions may perceive those conditions differently. When many travelers are surveyed, a distribution of responses from "very satisfied" to "very dissatisfied" (or some similar scale) results. The traveler-perception models predict the average of those responses.
5. Some alternative tools involve the use of simulation, in which results will vary as inputs are randomly varied within a set distribution and average. Therefore, reporting only one result from simulation simplifies the actual results produced.

Therefore, any reported traffic operations performance measure value, whether resulting from an HCM methodology, an alternative tool (e.g., simulation), or even field measurement, potentially has a fairly wide range associated with it in which the "true" value actually lies. The LOS concept helps to downplay the implied accuracy of a numeric result by presenting a range of measure results as being reasonably equivalent from a traveler's point of view. However, the same variability issues also mean that the "true" LOS value may be different from the one predicted by a methodology. One way of thinking about a reported value and its corresponding LOS result is that they are the statistical "best estimators" of conditions.

LOS Reported Separately by Mode

In an effort to produce a single top-level measure of conditions, some HCM users may be tempted to blend the LOS reported for each mode into a single LOS value for a roadway element. However, each mode's travelers have different perspectives and could experience different conditions while traveling along a particular roadway. The use of a blended LOS carries the risk of overlooking quality-of-service deficiencies for nonautomobile travelers that discourage the use of those modes, particularly if the blended LOS is weighted by the number of modal travelers. Other measures, such as person delay, can be used when an

analysis requires a combined measure. The HCM recommends reporting modal LOS results individually.

Reporting the Big Picture

Neither LOS nor any other single performance measure tells the full story of roadway performance.

Analysts and decision makers should always be mindful that neither LOS nor any other single performance measure tells the full story of roadway performance. Depending on the particulars of a given location and analysis, queue lengths, demand-to-capacity ratios, average travel speeds, indicators of safety, and other performance measures may be equally or even more important to consider, regardless of whether they are specifically called out in an agency standard. For this reason, the HCM provides methods for estimating a variety of useful roadway operations performance measures, and not just methods for determining LOS.

SERVICE MEASURES

Service measures are the performance measures that define LOS.

As introduced earlier, service measures are specific performance measures that are used to determine LOS. Exhibit 8-2 summarizes the service measures used by the HCM for different combinations of transportation system elements and travel modes. Some service measures are based on a traveler-perception model; the components of each model are given in Exhibit 8-3.

Exhibit 8-2
Service Measures by Individual System Element

System Element	Service Measures			
	Automobile	Pedestrian	Bicycle	Transit
Freeway facility	Density	--	--	--
Basic freeway segment	Density	--	--	--
Freeway weaving segment	Density	--	--	--
Ramp junction	Density	--	--	--
Multilane highway	Density	--	LOS score[a]	--
Two-lane highway	Percent time-spent-following, speed	--	LOS score[a]	--
Urban street facility	Speed	LOS score[a]	LOS score[a]	LOS score[a]
Urban street segment	Speed	LOS score[a]	LOS score[a]	LOS score[a]
Signalized intersection	Delay	LOS score[a]	LOS score[a]	--
Two-way stop	Delay	Delay	--	--
All-way stop	Delay	--	--	--
Roundabout	Delay	--	--	--
Interchange ramp terminal	Delay	--	--	--
Off-street pedestrian–bicycle facility	--	Space, events[b]	LOS score[a]	--

Notes: [a] See Exhibit 8-3 for the LOS score components.
[b] Events are situations where pedestrians meet bicyclists.

Exhibit 8-3
Components of Traveler-Perception Models Used to Generate Service Measures

System Element	Mode	Model Components
Multilane highway and two-lane highway	Bicycle	Perceived separation between bicycles and motor vehicles, pavement quality, automobile and heavy vehicle volume and speed
Urban street facility	Automobile	Weighted average of segment automobile LOS scores
	Pedestrian	Urban street segment and signalized intersection pedestrian LOS scores, midblock crossing difficulty
	Bicycle	Urban street segment and signalized intersection bicycle LOS scores, driveway conflicts
	Transit	Weighted average of segment transit LOS scores
Urban street segment	Automobile	Stops per mile, left-turn lane presence
	Pedestrian	Pedestrian density, sidewalk width, perceived separation between pedestrians and motor vehicles, motor vehicle volume and speed
	Bicycle	Perceived separation between bicycles and motor vehicles, pavement quality, automobile and heavy vehicle volume and speed
	Transit	Service frequency, perceived speed, pedestrian LOS
Signalized intersection	Pedestrian	Street crossing delay, pedestrian exposure to turning vehicle conflicts, crossing distance
	Bicycle	Perceived separation between bicycles and motor vehicles, crossing distance
Off-street pedestrian–bicycle facility	Bicycle	Average meetings/minute, active passings/minute, path width, centerline presence, delayed passings

Note: The automobile traveler-perception model for urban street segments and facilities is not used to determine LOS; however, it is provided as a performance measure to facilitate multimodal analyses.

4. ANALYSIS PROCESS

LEVELS OF HCM ANALYSIS

The HCM can be applied at the *operational, planning and preliminary engineering,* and *design* analysis levels. The required input data typically remain the same at each analysis level, but the degree to which default values are used instead of measured or forecast values differs. In addition, operational and planning and preliminary engineering analyses frequently evaluate the LOS that will result from a given set of inputs, while design analyses evaluate the facility characteristics that will be needed to achieve a desired LOS.

Operational Analysis

In an operational analysis, an analyst applies an HCM methodology directly and supplies all of the required input parameters from measured or forecast values. No, or minimal, default values are used. Of the available ways to apply HCM methodologies, operational analyses provide the highest level of accuracy but, as a result, also require the most detailed data collection, which has time and cost implications.

An operational analysis helps in making decisions about operating conditions. Typical alternatives consider, for example, changes in traffic signal timing and phasing, changes in lane configurations, spacing and location of bus stops, the frequency of bus service, or the addition of a bicycle lane. The analysis produces operational measures that can be used to compare the alternatives.

As discussed earlier in this chapter, even though a model's results may be highly accurate, any variability associated with the model's inputs can affect the model's results.

Planning and Preliminary Engineering Analysis

In planning and preliminary engineering analyses, an analyst applies an HCM methodology by using default values for some to nearly all of the model inputs—for example, through the use of generalized service-volume tables. The results are less accurate than in an operations analysis, but the use of default values reduces the amount of data collection and the time required to perform an analysis. In a large-scale planning study, where a large number of roadways may be evaluated, this level of analysis may be the best practical, given time and budget constraints. For future-focused studies, not all of the model inputs may be known or forecastable, which suggests the need for a planning analysis with the use of default values for the unknown model inputs.

Planning analyses are applications of the HCM generally directed toward broad issues such as initial problem identification (e.g., screening a large number of locations for potential operations deficiencies), long-range analyses, and statewide performance monitoring. An analyst often must estimate the future times at which the transportation system will fall below a desired LOS. Preliminary engineering analyses are often conducted to support planning decisions related to a roadway design concept and scope and in performing alternatives analyses. These studies can also assess proposed systemic policies,

such as lane-use control for heavy vehicles, systemwide freeway ramp metering and other intelligent transportation systems applications, and the use of demand-management techniques, such as congestion pricing (*5*).

Generalized Service-Volume Tables

Generalized service-volume tables are sometimes used in planning analyses. These tables are constructed by applying default values to an HCM methodology and then incrementally determining the maximum number of vehicles that a roadway could carry at a given LOS, given the assumed conditions.

Service-volume results should be applied with care, since actual conditions will likely vary in some way from the assumptions used to develop the table.

The use of a service-volume table is most appropriate in situations in which it is not practical to evaluate every roadway or intersection within a study area. Examples of these applications would be city, county, or statewide planning studies, where the size of the study area makes it infeasible to conduct a capacity or LOS analysis for every roadway segment. For these types of planning applications, the focus of the effort is simply to highlight potential problem areas (for example, locations where demand may exceed capacity or where a desired LOS may be exceeded). For such applications, a service-volume table can be a useful screening tool. Once potential problem areas have been identified, more detailed analyses can be performed for those locations.

The HCM provides generalized service-volume tables for

- *Freeway facilities*
- *Multilane highways*
- *Two-lane highways*
- *Urban street facilities*

The characteristics of any given roadway will likely vary in some way from the assumed input values used to develop a service-volume table. Therefore, the results from a service-volume table should be treated as rough approximations. Service-volume tables should not be substituted for other tools to make a final determination of the operational adequacy of a particular roadway.

Design Analysis

Design analyses typically apply the HCM to establish the detailed physical features that will allow a new or modified roadway to operate at a desired LOS. Design projects are usually targeted for mid- to long-term implementation. Not all the physical features that a designer must determine are reflected in the HCM models. Typically, analysts using the HCM are seeking to determine such elements as the basic number of lanes required and the need for auxiliary or turning lanes. However, an analyst can also use the HCM to establish values for elements such as lane width, steepness of grade, the length of added lanes, the size of pedestrian queuing areas, the widths of sidewalks and walkways, and the presence of bus pullouts.

The data required for design analyses are detailed and are based substantially on proposed design attributes. However, the intermediate- to long-term focus of the work will require the use of some default values. This simplification is justified in part by the limits on the accuracy and precision of the traffic forecasts with which the analyst will be working.

ANALYSIS TOOL SELECTION

Types of Tools

Each analytical or simulation tool, depending on the application, has its own strengths and weaknesses. It is important to relate relevant modeling features to the needs of the analysis and to determine which tool satisfies these needs to the greatest extent.

HCM methodologies are *deterministic* and *macroscopic.* A deterministic model will always produce the same result for a given set of inputs. A macroscopic model considers average conditions experienced by vehicles over a period of time (typically 15 min or 1 h). In comparison, microsimulation models are *stochastic* and *microscopic.* In a stochastic model, a different random number seed will produce a different modeling result; therefore, the outcome from a simulation run based on a stochastic model cannot be predicted with certainty before the analysis begins. Microscopic models simulate the movement of individual vehicles on the basis of car-following and lane-changing theories.

Situations When Alternative Tools Might Be Considered

The HCM is the product of a large number of peer-reviewed research projects and reflects the best available techniques (at the time of publication) for determining capacity and LOS. However, the research behind the HCM has not addressed every possible situation that can arise in the real world. Therefore, the HCM documents the limitations of its procedures and highlights situations when alternative analysis tools should be considered to supplement or substitute for the HCM. The following are examples of these situations:

- The configuration of the facility has elements that are beyond the scope of the HCM procedures. Each HCM procedural chapter identifies the specific limitations of its own methodology.
- Viable alternatives being considered in the study require the application of an alternative tool to make a more informed decision.
- The measures are compatible with corresponding HCM measures and the decision process requires additional performance measures, such as fuel consumption and emissions, that are beyond the scope of the HCM.
- The system under study involves a group of different facilities with mutual interactions that require the use of more than one HCM chapter. Alternative tools are able to analyze these facilities as a single system.
- Routing is an essential part of the problem being addressed.
- The quantity of input or output data required presents an intractable problem for the HCM procedures.
- The HCM procedures predict oversaturated conditions that last throughout a substantial part of a peak period or queues that overflow the available storage space.

The Federal Highway Administration's Traffic Analysis Toolbox (*6*) provides general guidance on the use of traffic analysis tools, including the HCM. More detailed guidance for alternative tool application to specific system elements is

presented in Volumes 2 and 3 of the HCM. Supplemental examples involving situations beyond the scope of the HCM procedures are presented in Volume 4.

INTERPRETING RESULTS

Uncertainty and Variability

Model outputs—whether from the HCM or alternative tools—are estimates of the "true" values that would be observed in the field. Actual values will lie within some range of the estimated value. The magnitude of the range, and therefore the degree of uncertainty, is a function of several variables, including the quality of the input data, the inherent variability of the model, and the degree to which the model accounts for all of the factors that govern the analysis. The uncertainty may be amplified to some extent by imperfect knowledge of the traveler-perception aspects of quality of service.

When stochastic simulation tools are used, uncertainty is normally addressed through multiple runs with different random number seeding. Regardless of the modeling approach, a sensitivity analysis may be performed to assess the degree to which input data variation is likely to affect the range of performance measure outputs. Depending on the model and the specifics of the situation being modeled, small changes in model inputs can have large impacts on model outputs.

Accuracy and Precision

Accuracy and precision are independent but complementary concepts. *Accuracy* relates to achieving a correct answer, while *precision* relates to the size of the estimation range of the parameter in question. In most cases, the field data on which the analyses are based can only be expected to be accurate to within 5% or 10% of the true value. Thus, the computations performed with these inputs cannot be expected to be extremely accurate, and the final results must be considered as estimates that are accurate and precise only within the limits of the inputs used.

HCM users should be aware of the limitations of the accuracy and precision of the methodologies in the manual. Such awareness will help users interpret the results of an analysis and use the results to make a decision on the design or operation of a transportation facility.

Comparing HCM Results with Alternative Tools

The exact definitions of performance measures are an important issue, particularly when a comparison of performance measures produced by different tools is to be made. Many tools produce performance measures with the same name (e.g., "delay"), but the definitions and methods of computation can differ widely. Chapter 7, Interpreting HCM and Alternative Tool Results, presents general guidance on comparing results. The procedural chapters in Volumes 2 and 3 present guidance on this topic for specific system elements.

Use of Vehicle Trajectory Analysis in Determining Performance Measures

It was pointed out in Chapter 4, Traffic Flow and Capacity Concepts, that the analysis of individual vehicle trajectories is becoming recognized as the most desirable method of producing performance measures that are consistent among different tools and compatible with the measures produced by the analysis procedures given in this manual. The HCM provides trajectory-based definitions and computational procedures for all of the commonly used performance measures. Alternative tool developers are encouraged to incorporate these definitions and computations into their products. Users of this manual are encouraged to consider the degree to which the performance measures comply with the HCM specifications in their traffic analysis tool selection process.

Temporal and Spatial Boundaries

Another source of difference in the performance measures obtained from different tools lies in their treatment of incomplete trips. Incomplete trips include those that enter a facility during a given analysis period and exit during a subsequent period and those that exit a facility after entering in a previous analysis period. To overcome differences among analysis tools, it is important that the analysis period cover enough time and space to include an uncongested interval at all boundaries.

When undersaturated operation is being studied, the definition of the facility in time and space is less important. The operation tends to be more homogeneous when d/c ratios are less than 1.00. For most performance measures, extending the analysis period will give a larger sample of vehicles, but it will not affect the performance measures significantly.

PRESENTING RESULTS

Tabular values and calculated results are displayed in a consistent manner throughout the HCM. It is suggested that analysts adhere to these conventions. A key objective is to use a number of significant digits indicating to users, decision makers, and other viewers that the results are not extremely precise but take on the precision and accuracy associated with the input variables used. This guidance applies primarily to inputs and final outputs; intermediate results in a series of calculations should not be rounded unless specifically indicated by a particular methodology.

Historically, data and analysis results have been presented primarily in tables. However, results may be best presented as pictures and only supplemented as necessary with the underlying numbers in some situations. Graphs and charts should not be used to decorate data or to make dull data entertaining; they should be conceived and fashioned to aid in the interpretation of the meaning behind the numbers.

5. DECISION-MAKING CONSIDERATIONS

The HCM provides procedures for capacity and quality-of-service analyses and therefore serves as an analytical tool for transportation engineers and planners. The 2010 edition of the HCM presents the best available techniques at the time of publication for determining roadway capacity and LOS, proven to work in the United States and validated by a group of independent experts. However, the HCM is only a guidance document: it does not endeavor to establish a legal standard for highway design or construction. This section describes the role of other guidance and standards documents that complement the HCM, along with issues for decision makers to consider should they choose to adopt HCM service measures as standards.

ROLE OF HCM COMPANION DOCUMENTS

Throughout its 60-year history, the HCM has been a fundamental reference work for transportation engineers and planners. However, it is but one of a number of documents that play a role in the planning, design, and operation of transportation facilities and services. The HCM's scope is to provide tools to evaluate the performance of highway and street facilities in terms of operational and traveler-perception measures. This section describes companion documents to the HCM that cover important topics outside the HCM's scope.

Highway Safety Manual

The *Highway Safety Manual* (HSM) (*7*) provides analytical tools and techniques for quantifying the safety effects of decisions related to planning, design, operations, and maintenance. The information in the HSM is provided to assist agencies as they integrate safety into their decision-making processes. It is a nationally used resource document intended to help transportation professionals conduct safety analyses in a technically sound and consistent manner, thereby improving decisions made on the basis of safety performance.

A Policy on Geometric Design of Highways and Streets

The American Association of State Highway and Transportation Officials' (AASHTO's) *A Policy on Geometric Design of Highways and Streets* ("Green Book") (*8*) provides design guidelines for roadways ranging from local streets to freeways, in both urban and rural locations. The guidelines "are intended to provide operational efficiency, comfort, safety, and convenience for the motorist," while also emphasizing the need to consider the use of roadway facilities by other modal users.

Manual on Uniform Traffic Control Devices

The Federal Highway Administration's *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (*9*) is the national standard for traffic control devices for any street, highway, or bicycle trail open to public travel. Of particular interest to HCM users are the sections of the MUTCD pertaining to warrants for all-way STOP control and traffic-signal control, signing and

markings to designate lanes at intersections, and associated considerations of adequate roadway capacity and less restrictive intersection treatments.

Transit Capacity and Quality of Service Manual

The *Transit Capacity and Quality of Service Manual* (TCQSM) (*10*) is the transit counterpart to the HCM. The TCQSM contains information on the various types of public transportation and provides a framework for measuring transit availability, comfort, and convenience from the passenger point of view. The manual contains quantitative techniques for calculating the capacity of bus, rail, and ferry transit services and of transit stops, stations, and terminals.

TOOLS VERSUS STANDARDS

Although the HCM does not set standards—for example, it does not specify a particular LOS that should be provided for a particular roadway type—it is referenced in the AASHTO Green Book (*8*), and numerous agencies and jurisdictions have adopted LOS standards based on the HCM. This section discusses issues that agencies and jurisdictions should consider when they adopt standards or guidance relating to LOS or default values used in HCM analyses.

Evolution of HCM Analysis Procedures

Each new edition of the HCM has incorporated new methodologies and—in some cases—new service measures for evaluating roadway system elements. This edition of the HCM is no different.

Changes in HCM methodologies are made to improve estimates of service and other performance measures. These types of changes do not directly affect an agency's adopted LOS standards, since the service measure for a particular roadway element and the threshold values for particular levels of service do not change. However, methodological changes frequently have indirect effects. For example, the estimated service measure value for a particular roadway might have met an agency's LOS standard under the previous methodology but might not meet the standard under the new methodology, and vice versa. Similarly, agencies that have adopted generalized service-volume tables may find that the service volumes associated with a given LOS have changed as a result of the methodological change. In this situation, the underlying estimate of the roadway's operation will not have changed, but the estimate of the traffic volume that produces that operation will have changed.

Even when the basic methodology for a roadway element does not change, the recommended default values for the methodology may have changed, which can also cause the issues described above with service-volume tables and with old and new methodologies producing different results. Following the HCM's recommendations of using field-measured input values whenever possible and locally generated default values otherwise avoids this issue.

A service measure's LOS thresholds or the measure itself may change from one edition of the HCM to the next. In that case, it is recommended that an agency that has adopted an operations standard for that measure revisit the standard to ensure that it still accurately represents the quality of service the agency wishes to provide. These kinds of changes in the HCM may also have

planning and project programming implications, as the need for or scale of a given project may change.

Incorporating HCM Analysis Results into Decision Making

Agencies and jurisdictions adopt roadway design and operations standards for a number of reasons, including consistency in roadway design across a jurisdiction and provision of an objective basis for making decisions on required improvements. As mentioned earlier, numerous agencies and jurisdictions have chosen to adopt LOS standards for their roadways. The existence of computerized tools that implement HCM procedures makes it easy for analysts to test a number of roadway improvements against an LOS standard. However, the analysis does not end once an LOS result has been determined.

The existence of a LOS F condition does not, by itself, indicate that action must be taken to correct the condition. Conversely, meeting a LOS standard does not necessarily mean that no problem exists or that an improvement that produces the desired LOS is a desirable solution. Other issues, including but not limited to safety, impacts on other modes, traffic-signal warrants, turn-lane warrants, cost–benefit issues, and access management, may also need to be considered as part of the analysis, recommendations, and eventual decision. As always, engineering judgment should be applied to any recommendations resulting from HCM (or alternative tool) analyses.

Two examples of common situations where a LOS result considered by itself might lead to a decision different from one that would be reached if other factors were also considered are given below.

Traffic Signal Warrants

The MUTCD (*9*) provides a number of warrants that indicate when a traffic signal may be justified. It is possible to have a condition at a two-way STOP intersection—particularly when a low-volume minor street intersects a very high-volume major street—where the minor street approach operates at LOS F but does not meet traffic signal warrants. Because the MUTCD is the standard for determining when a traffic signal is warranted, a LOS F condition by itself is not sufficient justification for installing a signal.

Turn-Lane Warrants

A number of agencies and jurisdictions have adopted warrants that indicate when the installation of turn lanes may be justified at an intersection. It is possible for an HCM analysis to indicate that the addition of a turn lane will result in an acceptable LOS but for the turn-lane warrant analysis to determine that the necessary conditions for installing a turn lane have not been satisfied. In this case, the potential for a satisfactory LOS in the future would not be sufficient justification by itself for installing the turn lane.

6. REFERENCES

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CHAPTER 9
GLOSSARY AND SYMBOLS

CONTENTS

1. GLOSSARY

Chapter 9, Glossary and Symbols, defines the terms used in the *Highway Capacity Manual* (HCM) and presents the symbols and abbreviations used in the manual. Highway transportation terminology has evolved over time to create multiple definitions, and the confusion has been compounded by technical jargon. The definitions, abbreviations, and symbols presented here are intended to establish a consistent terminology for use in the HCM. It is recognized that other definitions and usage could exist.

A

Acceleration lane – A paved noncontinuous lane, including tapered areas, allowing vehicles to accelerate when they enter the through-traffic lane of the roadway.

Acceleration/deceleration delay – Delay experienced by vehicles slowing from and subsequently returning to their running speed.

Access point – An intersection, driveway, or opening on either side of a roadway.

Access-point density – The total number of access points on both sides of the roadway, divided by the length of the segment.

Accessibility – The percentage of the populace able to complete a selected trip within a specified time.

Accuracy – The degree of an estimate's agreement with a standard or true value.

Active passings – The number of other path users traveling in the same direction as an average bicyclist (i.e., a bicyclist traveling at the average speed of all bicycles) who are passed by that bicyclist.

Active priority – A form of traffic signal priority that adjusts signal timing in reaction to the arrival of a bus.

Active traffic management (ATM) – The dynamic and continuous monitoring and control of traffic operations on a facility to improve its performance.

Actuated control – Consists of a defined phase sequence in which the presentation of each phase is dependent on whether the phase is on recall or the associated traffic movement has submitted a call for service through a detector.

Adjustment – An additive or subtractive quantity that adjusts a parameter for a base condition to represent a prevailing condition.

Adjustment factor – A factor that adjusts a parameter for a base condition to represent a prevailing condition.

Aggregate delay – The summation of delays for multiple lanes or lane groups, usually aggregated for an approach, an intersection, or an arterial route.

Algorithm – A set of rules for solving a problem in a finite number of steps.

All-way STOP-controlled (AWSC) intersection – An intersection with STOP signs on all approaches. The driver's decision to proceed is based on a consensus of right-of-way governed by the traffic conditions of the other approaches and the rules of the road (e.g., the driver on the right has the right-of-way if two vehicles arrive simultaneously).

Alternative tool – An analysis procedure outside of the HCM that may be used to compute measures of transportation system performance for analysis and decision support.

Analysis hour – A single hour for which a capacity analysis is performed on a system element.

Analysis period – A single time period (for example, the peak 15 min of the peak hour) during which a capacity analysis is performed on a system element.

Analytical model – A model based on traffic flow theory, combined with the use of field measures of driver behavior, resulting in an analytic formulation of the relationship between the field measures and performance measures such as capacity and delay.

Annual average daily traffic (AADT) – The total volume of traffic passing a point or segment of a highway facility in both directions for 1 year divided by the number of days in the year.

Approach – A set of lanes at an intersection that accommodates all left-turn, through, and right-turn movements from a given direction.

Approach delay – The control delay for a given approach.

Approach grade – The average grade along the approach, as measured from the stop line to a point 100 ft upstream of the stop line along a line parallel to the direction of travel. An uphill condition has a positive grade, and a downhill condition has a negative grade.

Area – An interconnected set of transportation facilities serving movements within a specified geographic space, as well as movements to and from adjoining areas.

Area type – A description of the environment in which a system element is located.

Arrival rate – The mean of the statistical distribution of vehicles arriving at a point or uniform segment of a lane or roadway.

Arrival type – Six assigned categories for the quality of progression for a given approach to a signalized intersection.

Arterial street – A street interrupted by traffic control devices (e.g., signals, STOP signs, or YIELD signs) that primarily serves through traffic and that secondarily provides access to abutting properties. See also *urban street.*

Automobile mode – A travel mode that includes all motor vehicle traffic using a roadway except transit buses. It includes such vehicles as trucks, recreational vehicles, motorcycles, and tour buses.

Auxiliary lane – See *freeway auxiliary lane.*

Available time-space – The product of available time and available space for pedestrian circulation on a crosswalk at a signalized intersection.

Average bus stop spacing – The average distance between bus stops along a street.

Average grade – The total rise from the beginning of the composite grade to the point of interest, divided by the length of the grade (to the point of interest).

Average running speed – The length of a segment divided by the average running time of vehicles that traverse the segment.

Average spot speed – See *time mean speed.*

Average travel speed – The length of the highway segment divided by the average travel time of all vehicles traversing the segment, including all stopped delay times. Equal to *space mean speed.*

B

Back of queue – The maximum backward extent of queued vehicles during a typical cycle, as measured from the stop line to the last queued vehicle.

Barrier – 1. A separation of intersecting movements in distinct rings to prevent operating conflicting phases at the same time. **2.** A physical object or pavement marking designed to prevent vehicles from entering or departing a section of roadway.

Barrier pair – A pair of phases within the same ring and barrier that cannot be displayed concurrently.

Base conditions – A set of specified standard conditions (e.g., good weather, good and dry pavement conditions, familiar users, no impediments to traffic flow) that must be adjusted to account for prevailing conditions that do not match.

Base length – The distance between the points in a weaving segment where the edges of the travel lanes of the merging and diverging roadways converge.

Base saturation flow rate – The saturation flow rate under base conditions.

Baseline uniform delay – The average uniform delay when there is no initial queue.

Basic freeway segment – A length of freeway facility whose undersaturated operations are unaffected by weaving, diverging, or merging.

Bicycle – A vehicle with two wheels tandem, propelled by human power, and usually ridden by one person.

Bicycle facility – A road, path, or way specifically designated for bicycle travel, whether exclusively or with other vehicles or pedestrians.

Bicycle lane – A portion of a roadway designated by striping, signing, and pavement markings for the preferential or exclusive use of bicycles.

Bicycle mode – A travel mode under which a nonmotorized bicycle is used on a roadway or pathway.

Bicycle path – A bikeway physically separated from motorized traffic by an open space or barrier, either within the highway right-of-way or within an independent right-of-way.

Bicycle speed – The riding speed of bicycles, in miles per hour or feet per second.

Bicycle track – A one-way bicycle facility separated from both motor vehicle traffic and the sidewalk by low curbs.

Body ellipse – The practical minimum area for standing pedestrians.

Bottleneck – A system element on which demand exceeds capacity.

Boundary intersection – An intersection defining the endpoint of an urban street segment.

Breakdown – The transition from noncongested to congested conditions typically observed as a speed drop accompanied by queue formation.

Breakdown flow – The flow at which operations transition from noncongested to congested.

Buffer width – The distance between the outside edge of the paved roadway (or face of curb, if present) and the near edge of the sidewalk.

Bus – A self-propelled, rubber-tired road vehicle designed to carry a substantial number of passengers (at least 16) and commonly operated on streets and highways.

Bus lane – A highway or street lane reserved primarily for buses during specified periods. It may be used by other traffic under certain circumstances, such as making a right or left turn, or by taxis, motorcycles, or carpools that meet the requirements of the jurisdiction's traffic laws.

Bus mode – A transit mode operated by rubber-tired vehicles that follow fixed routes and schedules along roadways.

Bus shelter – A structure with a roof and (typically) three enclosed sides that protects waiting transit passengers from wind, rain, and sun.

Bus stop – A designated area along a street where one or more buses can simultaneously stop to load and unload passengers. It can be on-line (buses stop wholly or partially in the travel lane) or off-line (buses stop out of the travel lane).

Busway – A right-of-way restricted to buses by a physical separation from other traffic lanes.

Bypass lane – A lane provided at a roundabout that allows a particular traffic movement to avoid using the circulatory roadway.

C

Calibration – The process by which the analyst modifies model parameters so that the model estimates best reproduce field-measured local traffic conditions.

Call – A registration of a demand for right-of-way by vehicles or pedestrians to a controller.

Capacity – The maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions.

Case – See *degree-of-conflict case.*

Central area pricing – An areawide implementation of congestion pricing that imposes tolls for vehicles both entering and traveling within a central area street network during certain hours of certain days.

Central business district (CBD) – An area with characteristics including narrow street rights-of-way, frequent parking maneuvers, vehicle blockages, taxi and bus activity, small-radius turns, limited use of exclusive turn lanes, high pedestrian activity, dense population, and midblock curb cuts.

Change interval – See *yellow change interval.*

Circulating flow – The flow conflicting with the entry flow on the subject approach to a roundabout (i.e., the flow passing in front of the splitter island next to the subject entry).

Circulation area – 1. The portion of a sidewalk intended to be used for pedestrian movement. **2.** The average area available to each person using a pedestrian facility.

Circulation time-space – The total available time-space minus the time-space occupied by pedestrians waiting to cross a crosswalk.

Circulatory roadway – The continuous-flow section of a roundabout that requires other vehicles entering the roadway to yield.

Class I two-lane highways – Highways where motorists expect to travel at relatively high speeds, such as major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks.

Class II two-lane highways – Highways where motorists do not necessarily expect to travel at high speeds, such as access routes to Class I facilities, scenic or recreational routes, or routes passing through rugged terrain.

Class III two-lane highways – Highways serving moderately developed areas, such as portions of a Class I or Class II highway that pass through small towns or developed recreational areas.

Clearance interval – See *red clearance interval.*

Clearance lost time – The time at the end of a signal phase during which the movements served by that phase are not used by any traffic because drivers decelerate and stop in response to the presentation of a yellow indication.

Clearance time – 1. The interval after a bus is ready to depart during which a loading area is not available for use by a following bus, consisting of the sum of reentry delay and the time for a bus to start up and travel its own length, clearing the stop. **2.** See *clearance lost time* and *red clearance interval.*

Climbing lane – A passing lane added on an upgrade to allow traffic to pass heavy vehicles whose speeds are reduced.

Cloverleaf interchange – An interchange with four loop ramps and four diagonal ramps, with no traffic control on either crossing roadway.

Collector street – A surface street providing land access and traffic circulation within residential, commercial, and industrial areas.

Collector–distributor road (C-D road) – A continuous roadway without local access provided parallel to a freeway mainline through one or more interchanges for the purpose of removing weaving movements or closely spaced merges and diverges from the mainline.

Common green time – The period of time when the phases at the two intersections of an interchange both provide a green indication to a particular origin–destination movement.

Complete trip – A vehicle that enters the spatial domain of an analysis during the analysis period and is able to exit the domain successfully before the end of the analysis period.

Composite grade – A series of adjacent grades along a highway that cumulatively has a more severe effect on operations than each grade separately.

Compressed diamond interchange – A diamond interchange with a separation of 400 to 800 ft between the two intersections.

Computational engine – A software implementation of one or more models.

Conflict – The crossing, merging, or diverging of two traffic movements at an intersection.

Conflicting approach – At an all-way STOP-controlled intersection, an approach to the left or right of the subject approach.

Conflicting flow rate – The total flow rate in conflict with a specific movement at an unsignalized intersection.

Conflicting movements – Vehicular, pedestrian, or bicycle streams that seek to occupy the same space at the same time.

Congestion – 1. Low-flow, high-occupancy traffic operations that arise when demand approaches or exceeds a system element's capacity. **2.** A difference between the highway system performance expected by users and how the system actually performs—for example, an intersection that may seem very congested in a rural community may not even register as an annoyance in a large metropolitan area.

Congestion pricing – The practice of charging tolls for use of all or part of a facility or a central area according to the severity of congestion.

Continuous-flow interchange – A diamond interchange form where left-turning traffic from the arterial street to an on-ramp is crossed to the left side of the roadway in advance of the interchange. Between the crossover and the near ramp terminal, the left-turning traffic travels in its own roadway between the opposing through arterial street traffic (on its right) and right-turning traffic from the off-ramp (on its left). The left-turning traffic continues straight at the near ramp terminal and turns left onto the on-ramp at the far ramp terminal without conflict from opposing through vehicular traffic. Also known as a displaced left-turn interchange.

Control condition – The traffic controls and regulations in effect for a segment of street or highway, including the type, phasing, and timing of traffic signals; STOP signs; lane use and turn controls; and similar measures.

Control delay – Delay associated with vehicles slowing in advance of an intersection, the time spent stopped on an intersection approach, the time spent as vehicles move up in the queue, and the time needed for vehicles to accelerate to their desired speed.

Controlled – Having a traffic control device that interrupts traffic flow (e.g., a traffic signal, STOP sign, or YIELD sign).

Conventional diamond interchange – A diamond interchange with a separation of 800 ft or more between the two intersections.

Coordinated actuated control – A variation of semiactuated operation that uses the controller's force-off settings to constrain the noncoordinated phases associated with the minor movements such that the coordinated phases are served at the appropriate time during the signal cycle and progression for the major movements is maintained.

Corridor – A set of parallel transportation facilities, for example a freeway and an arterial street.

Crawl speed – The maximum sustained speed that can be maintained by a specified type of vehicle on a constant upgrade of a given percent.

Critical density – The density at which capacity occurs for a given facility.

Critical headway – The minimum headway in the major traffic stream that will allow the entry of one minor-street vehicle.

Critical lane groups – The lane groups that have the highest flow ratio for a given signal phase.

Critical phase – One phase of a set of phases that occur in sequence and whose combined flow ratio is the largest for the signal cycle.

Critical segment – The segment that will break down first, given that all traffic, roadway, and control conditions do not change, including the spatial distribution of demands on each component segment.

Critical speed – The speed at which capacity occurs for a segment.

Critical volume-to-capacity ratio – The proportion of available intersection capacity used by vehicles in critical lane groups.

Cross flow – A pedestrian flow that is approximately perpendicular to and crosses another pedestrian stream. The smaller of the two flows is the cross-flow condition.

Crossing time – The curb-to-curb crossing distance divided by the pedestrian walking speed specified in the *Manual on Uniform Traffic Control Devices*.

Crosswalk – See *pedestrian crosswalk.*

Crosswalk occupancy time – The product of the pedestrian service time and the number of pedestrians using a crosswalk during one signal cycle.

Curb extension – An extension of the sidewalk to the edge of the travel or bicycle lane.

Cycle – A complete sequence of signal indications.

Cycle failure – A condition where one or more queued vehicles are not able to depart an intersection as a result of insufficient capacity during the cycle in which they arrive.

Cycle length – The time elapsed between the endings of two sequential terminations of a given interval. For coordinated signals, this is measured by using the coordinated phase green interval.

Cyclic spillback – A condition where the downstream boundary intersection is signalized and its queue backs into the upstream intersection as a result of queue growth during the red indication.

D

***D*-factor –** The proportion of traffic moving in the peak direction of travel on a given roadway during the peak hour.

Daily service volume – The maximum total daily volume in both directions that can be sustained in a given segment without violating the criteria for LOS *i* in the peak direction in the worst 15 min of the peak hour under prevailing roadway, traffic, and control conditions.

Deceleration lane – A paved noncontinuous lane, including tapered areas, allowing vehicles leaving the through-traffic lane of the roadway to decelerate.

Default value – A representative value that may be appropriate in the absence of local data.

Degree-of-conflict case – For all-way STOP-controlled intersections, a particular combination of vehicle presence on other approaches with respect to the subject approach.

Degree of saturation – See *demand-to-capacity ratio.*

Delay – Additional travel time experienced by a driver, passenger, bicyclist, or pedestrian beyond that required to travel at the desired speed.

Delayed passing maneuver – The inability of an average bicyclist to make a passing

maneuver immediately due to the presence of both another path user ahead of the overtaking average bicyclist in the subject direction and a path user in the opposing direction.

Demand – The number of vehicles or other roadway users desiring to use a given system element during a specific time period, typically 1 h or 15 min.

Demand flow rate – The count of vehicles arriving at the system element during the analysis period, converted to an hourly rate. When measured in the field, this flow rate is based on a traffic count taken upstream of the queue associated with the system element. This distinction is important for counts made during congested periods because the count of vehicles departing the system element will produce a demand flow rate that is lower than the true rate.

Demand starvation – A condition occurring when a signalized approach has adequate capacity but a significant portion of the traffic demand is held upstream and cannot use the capacity provided because of the signalization pattern.

Demand volume – The number of vehicles that arrive to use the facility. Under noncongested conditions, demand volume is equal to the observed volume.

Demand-to-capacity ratio – The ratio of demand volume to capacity for a system element.

Density – The number of vehicles occupying a given length of a lane or roadway at a particular instant. See also *pedestrian density.*

Departure headway – The average time between departures of successive vehicles on a given approach at an all-way STOP-controlled intersection.

Descriptive model – A model that shows how events unfold given a logic that describes how the objects involved will behave.

Design hour – An hour with a traffic volume that represents a reasonable value for designing the geometric and control elements of a facility.

Design hour factor – The proportion of the AADT that occurs during the design or analysis hour. See also *K-factor.*

Design-level analysis – An application of the HCM to establish the detailed physical features that will allow a new or modified facility to operate at a desired LOS or performance value.

Design speed – A speed used to design the horizontal and vertical alignments of a highway.

Detection zone – The portion of a signalized intersection approach where a vehicle can be detected by the signal controller (with use of in-pavement loops or other technology), resulting in the display of the green indication for the approach being extended.

Deterministic model – A mathematical model that is not subject to randomness. For a given set of inputs, the result from the model is the same with each application.

Deterministic queue delay – The component of control delay representing the delay to all vehicles arriving during the analysis period when the demand flow rates exceed the capacity of an approach, computed by accumulating the vehicular inputs and outputs under the assumption of uniform arrivals.

Diamond interchange – An interchange form where one diagonal connection is made for each freeway entry and exit, with one connection per quadrant.

Directional design hour volume – The traffic volume for the design hour in the peak direction of flow.

Directional distribution – A characteristic of traffic that volume may be greater in one direction than in the other during any particular hour on a highway. See also *D-factor.*

Directional flow rate – The flow rate of a highway in one direction.

Directional segment – A length of two-lane highway in one travel direction with homogeneous cross sections and relatively constant demand volume and vehicle mix.

Directional split – See *D-factor.*

Diverge – A movement in which a single stream of traffic separates into two streams without the aid of traffic control devices.

Diverge segment – See *freeway diverge segment.*

Diverging diamond interchange – A diamond interchange form where through traffic on the arterial switches sides of the street at each of the ramp terminals, allowing left turns to ramps to be made without conflict from opposing through vehicular traffic.

Double-crossover interchange – See *diverging diamond interchange.*

Downstream – The direction of traffic flow.

Driver population factor – A parameter that accounts for driver characteristics that differ from base conditions and their effects on traffic.

Dual entry – A mode of operation (in a multiring controller) in which one phase in each ring must be in service. If a call does not exist in a ring when it crosses the barrier, a phase is selected in that ring to be activated by the controller in a predetermined manner.

Duration of congestion – The amount of time that congestion persists within a transportation system.

Dwell time – The sum of the time required to serve passengers at a transit stop and the time required to open and close the vehicle doors.

Dwell time variability – The distribution of dwell times at a stop because of fluctuations in passenger demand for buses and routes.

Dynamic traffic assignment model – A descriptive model that is based on an objective (e.g., minimize the travel time or disutility associated with a trip) that is gradually improved over a sequence of iterations until the network reaches a state of equilibrium.

E

Effective available time-space – The available crosswalk time-space, adjusted to account for the effect turning vehicles have on pedestrians.

Effective green time – The time during which a given traffic movement or set of movements may proceed at the saturation flow rate; it is equal to the split time minus the lost time.

Effective red time – The time during which a given traffic movement or set of movements is directed to stop; it is equal to the cycle length minus the effective green time.

Effective walk time – The time that a WALK indication is displayed to a crosswalk, plus the portion of the DON'T WALK indication used by pedestrians to initiate their crossing.

Effective walkway width – The portion of a pedestrian facility's width that is usable for pedestrian circulation.

85th percentile speed – A speed value that is exceeded by 15% of the vehicles in a traffic stream.

Empirical model – A model that describes system performance and that is based on the statistical analysis of field data.

Entrance ramp – See *on-ramp.*

Entry flow – The traffic flow entering a roundabout on the subject approach.

Environmental conditions – Conditions such as adverse weather, bright sunlight directly in drivers' eyes, and abrupt transitions from light to dark (such as at a tunnel entrance on a sunny day) that may cause drivers to slow down and increase their spacing, resulting in a drop in a roadway's capacity.

Event – A meeting or a passing on a shared-use path or bicycle facility.

Event-based model – A simulation model that advances from one event to the next, skipping over intervening points in time when no event occurs.

Excess wait time – The average number of minutes transit passengers must wait at a stop past the scheduled departure time.

Exclusive bus lane – A highway or street lane reserved for buses.

Exclusive off-street bicycle paths – Paths physically separated from highway traffic provided for the exclusive use of bicycles.

Exclusive turn lane – A designated left- or right-turn lane used only by vehicles making those turns.

Exit flow – The traffic flow exiting a roundabout to the subject leg.

Exit ramp – See *off-ramp.*

Extent of congestion – The maximum geographic length of the congestion on a transportation system at any one time.

F

Facility – A length of roadway, bicycle path, or pedestrian walkway composed of a connected series of points and segments.

Failure rate – The probability that a bus will arrive at a bus stop and find all available loading areas already occupied by other buses.

Far-side stop – A transit stop where transit vehicles cross an intersection before stopping to serve passengers.

Fixed-object effective width – The sum of the physical width of a fixed object along a

walkway or sidewalk, any functionally unusable space associated with the object, and the buffer given it by pedestrians.

Fixed obstruction – A nonmovable object along a roadway, including light poles, signs, trees, abutments, bridge rails, traffic barriers, and retaining walls.

Flared approach – At two-way STOP-controlled intersections, a shared right-turn lane that allows right-turning vehicles to complete their movement while other vehicles are occupying the lane.

Flow rate – The equivalent hourly rate at which vehicles or other roadway users pass over a given point or section of a lane or roadway during a given time interval of less than 1 h, usually 15 min.

Flow ratio – The ratio of the actual flow rate to the saturation flow rate for a lane group at an intersection.

Follow-up headway – The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street.

Follower density – The number of followers per mile per lane; the following state is defined as a condition in which a vehicle is following its leader by no more than 3 s.

Force-off – A point within a cycle where an actuated phase must end regardless of continued demand. These points in a coordinated cycle ensure that the coordinated phases are provided a minimum amount of green time.

Four-phase pattern – A type of operation at an all-way STOP-controlled intersection with multilane approaches, where drivers from a given approach enter the intersection together, as right-of-way passes from one approach to the next and each is served in turn.

Free flow – A flow of traffic unaffected by upstream or downstream conditions.

Free-flow speed – 1. The theoretical speed when the density and flow rate on a study segment are both zero. **2.** The prevailing speed on freeways at flow rates between 0 and 1,000 passenger cars per hour per lane (pc/h/ln).

Freeway – A fully access-controlled, divided highway with a minimum of two lanes (and frequently more) in each direction.

Freeway auxiliary lane – An additional lane on a freeway to connect an on-ramp and an off-ramp.

Freeway diverge segment – A freeway segment in which a single traffic stream divides to form two or more separate traffic streams.

Freeway facility – An extended length of freeway composed of continuously connected basic freeway, weaving, merge, and diverge segments.

Freeway facility capacity – The capacity of the critical segment among those segments composing a defined freeway facility.

Freeway merge segment – A freeway segment in which two or more traffic streams combine to form a single traffic stream.

Freeway section – A portion of a freeway facility with a constant demand and a constant number of lanes.

Freeway weaving segment – Freeway segments in which two or more traffic streams traveling in the same general direction cross paths along a significant length of freeway without the aid of traffic control devices (except for guide signs).

Full stop – The slowing of a vehicle to less than 5 mi/h.

Fully actuated control – A signal operation in which vehicle detectors at each approach to the intersection control the occurrence and length of every phase.

Furniture zone – The portion of the sidewalk between the curb and the area reserved for pedestrian travel; it may be used for landscaping, utilities, or pedestrian amenities.

G

Gap – The space or time between two vehicles, measured from the rear bumper of the front vehicle to the front bumper of the second vehicle. See also *headway*.

Gap acceptance – The process by which a driver accepts an available gap in traffic to perform a maneuver.

Gap out – A type of actuated operation for a given phase where the phase terminates because of a lack of vehicle calls within the passage time.

General terrain – An extended length of highway containing a number of upgrades and downgrades where no single grade is

long enough or steep enough to have a significant impact on the operation of the overall segment.

Generalized service volume table – A sketch-planning tool that provides an estimate of the maximum volume a system element can carry at a given level of service, given a default set of assumptions about the system element.

Geometric condition – The spatial characteristics of a facility, including approach grade, the number and width of lanes, lane use, and parking lanes.

Geometric delay – Delay caused by geometric features causing vehicles to reduce their speed in negotiating a system element.

Gore area – The area located immediately between the left edge of a ramp pavement and the right edge of the roadway pavement at a merge or diverge area.

Green time – The duration of the green indication for a given movement at a signalized intersection.

Green time (*g*/*C*) ratio – The ratio of the effective green time of a phase to the cycle length.

Growth factor – A percentage increase applied to current traffic demands to estimate future demands.

H

Hard conversion – The conversion of a value from U.S. customary units to metric units (or vice versa) through the application of rounding, such as from 12 ft to 3.6 m. See also *soft conversion.*

Headway – The time between two successive vehicles as they pass a point on the roadway, measured from the same common feature of both vehicles (for example, the front axle or the front bumper).

Heavy vehicle – A vehicle with more than four wheels touching the pavement during normal operation.

High-occupancy vehicle (HOV) – A vehicle with a defined minimum number of occupants (>1); HOVs often include buses, taxis, and carpools, when a lane is reserved for their use.

Highway – A general term for denoting a public way for purposes of vehicular travel, including the entire area within the right-of-way.

Hindrance – Discomfort and inconvenience to a bicyclist as a result of meeting, passing, or being overtaken by other pathway users.

Holding area waiting time – The average time that pedestrians wait to cross the street in departing from the subject corner.

Hybrid models – Models used with very large networks that apply microscopic modeling to critical subnetworks and mesoscopic or macroscopic modeling to the connecting facilities.

I

Impedance – The reduction in the capacity of lower-rank movements caused by the congestion of higher-rank movements at a two-way STOP-controlled intersection.

Incident – Any occurrence on a roadway that impedes the normal flow of traffic.

Incident delay – The component of delay that results from an incident, compared with the no-incident condition.

Incomplete trip – A vehicle that is unable to enter and exit successfully the spatial domain of an analysis within the analysis period.

Incremental delay – The second term of lane group control delay, accounting for delay due to the effect of random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity (i.e., cycle failure) and delay due to sustained oversaturation during the analysis period.

Influence area – 1. The base length of a freeway weaving segment plus 500 ft upstream of the entry point to the weaving segment and 500 ft downstream of the exit point from the weaving segment; entry and exit points are defined as the points where the appropriate edges of the merging and diverging lanes meet. **2.** The acceleration lane(s) and Lanes 1 and 2 of the freeway mainline for a distance of 1,500 ft downstream of the merge point. **3.** The deceleration lane(s) and Lanes 1 and 2 of the freeway for a distance of 1,500 ft upstream of the diverge point.

Initial queue – The unmet demand at the beginning of an analysis period, either observed in the field or carried over from the computations of a previous analysis period.

Initial queue delay – The third term of lane group control delay, accounting for delay due to a residual queue identified in a previous analysis period and persisting at the start of the current analysis period. This

delay results from the additional time required to clear the initial queue.

Inputs – The data required by a model.

Instantaneous acceleration – An acceleration determined from the relative speeds of a vehicle at time t and time $t - \Delta t$, assuming a constant acceleration during Δt.

Instantaneous speed – A speed determined from the relative positions of a vehicle at time t and time $t - \Delta t$, assuming a constant acceleration during Δt.

Intelligent transportation system (ITS) – Transportation technology that allows drivers and traffic control system operators to gather and use real-time information to improve vehicle navigation, roadway system control, or both.

Intensity of congestion – The amount of congestion experienced by users of a system.

Interchange – A system of interconnecting roadways providing for traffic movement between two or more highways that do not intersect at grade.

Interchange density – The average number of interchanges per mile, measured over 3 mi upstream and 3 mi downstream from the midpoint of the weaving segment.

Interchange ramp terminal – A junction of a ramp with a surface street serving vehicles entering or exiting a freeway.

Internal link – The segment between two signalized intersections at an interchange ramp terminal.

Interrupted flow – Traffic flow on facilities characterized by traffic signals, STOP signs, YIELD signs, or other fixed causes of periodic delay or interruption to the traffic stream.

Intersection delay – The total additional travel time experienced by drivers, passengers, or pedestrians as a result of control measures and interaction with other users of the facility, divided by the volume departing from the corresponding cross section of the facility.

Interval – A period of time in which all traffic signal indications remain constant.

Isolated intersection – An intersection experiencing negligible influence from upstream signalized intersections, where flow is effectively random over the cycle and without a discernible platoon pattern evident in the cyclic profile of arrivals.

J

Jam density – The maximum density that can be achieved on a segment. It occurs when speed is zero (i.e., when there is no movement of persons or vehicles).

K

***K*-factor –** The proportion of AADT that occurs during the peak hour. See also *design hour factor*.

L

Lane 1 – The highway lane adjacent to the outside shoulder.

Lane 2 – The highway lane adjacent to and left of Lane 1.

Lane balance – A condition when the number of lanes leaving a diverge point is equal to the number of lanes approaching it, plus one.

Lane distribution – A parameter used when two or more lanes are available for traffic in a single direction and the volume distribution varies between lanes, depending on traffic regulation, traffic composition, speed and volume, the number of and location of access points, the origin–destination patterns of drivers, the development environment, and local driver habits.

Lane group – A lane or set of lanes designated for separate analysis.

Lane group delay – The control delay for a given lane group.

Lane utilization – The distribution of vehicles among lanes when two or more lanes are available for a movement. See also *prepositioning*.

Lane width – The lateral distance between stripes for a given lane.

Lateral clearance – 1. The total left- and right-side clearance from the outside edge of travel lanes to fixed obstructions on a multilane highway. **2.** The right-side clearance distance from the rightmost travel lane to fixed obstructions on a freeway.

Leg – A set of lanes at an intersection accommodating all approaching movements to and departing movements from a given direction.

Level of service (LOS) – A quantitative stratification of a performance measure or measures that represent quality of service, measured on an A–F scale, with LOS A representing the best operating conditions

from the traveler's perspective and LOS F the worst.

Level of service score (LOS score) – A numerical output from a traveler perception model that typically indicates the average rating that travelers would give a transportation facility or service under a given set of conditions.

Level terrain – Any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars, typically containing short grades of no more than 2%.

Light rail mode – A transit mode operated by vehicles that receive power from overhead wires and that run on tracks that can be located at grade within street rights-of-way. See also *streetcar mode.*

Link – A length of roadway between two nodes or points.

Link length – The urban street segment length minus the width of the upstream boundary intersection.

Load factor – The number of passengers occupying a transit vehicle divided by the number of seats on the vehicle.

Loading area – A curbside space where a single bus can stop to load and unload passengers; bus stops include one or more loading areas.

Local street – A street that primarily serves a land-access function.

Local transit service – Transit service making regular stops along a street (typically every 0.25 mi or less).

Loop ramp – A ramp requiring vehicles to execute a left turn by turning right, accomplishing a 90-degree left turn by making a 270-degree right turn.

Lost time – The time during which a movement's phase is active and the approach is not used effectively by that movement; it is the sum of clearance lost time and start-up lost time.

M

Macroscopic model – A mathematical model that considers traffic operations averaged over specified time intervals and specified segments or links without recognizing individual vehicles in the traffic stream.

Mainline – The primary through roadway as distinct from ramps, auxiliary lanes, and collector–distributor roads.

Mainline output – The maximum number of vehicles that can exit a freeway node, constrained by downstream bottlenecks or by merging traffic.

Major diverge area – A junction where one freeway segment diverges to form two primary freeway segments with multiple lanes.

Major merge area – A junction where two primary freeway segments, each with multiple lanes, merge to form a single freeway segment.

Major street – The street not controlled by STOP signs at a two-way STOP-controlled intersection.

Major weaving segment – A weaving segment with at least three entry and exit legs, each with two or more lanes.

Managed lanes – A limited number of lanes set aside within a freeway cross section where multiple operational strategies are utilized and actively adjusted as needed for the purpose of achieving predefined performance objectives. Examples include priced lanes and special-use lanes such as high-occupancy vehicle, express, bus-only, or truck-only lanes.

Max out – A type of actuated operation for a given phase where the phase terminates because it has reached the designated maximum green time for the phase.

Maximum green – The maximum amount of time that a green signal indication can be displayed in the presence of conflicting demand.

Maximum recall – A form of phase recall where the controller places a continuous call for vehicle service on the phase. This results in the presentation of the green indication for its maximum duration every cycle.

Maximum weaving length – The length at which weaving turbulence no longer has an impact on the capacity of the weaving segment.

Median – The area in the middle of a roadway separating opposing traffic flows.

Meeting – An encounter of bicycles or pedestrians moving in the opposite direction of the subject bicycle flow.

Merge – A movement in which two separate streams of traffic combine to form a single

stream without the aid of traffic signals or other right-of-way controls.

Merge segment – See *freeway merge segment.*

Mesoscopic model – A mathematical model for the movement of clusters or platoons of vehicles incorporating equations to indicate how the clusters interact.

Microscopic model – A mathematical model that captures the movement of individual vehicles and their car-following, lane-choice, and gap-acceptance decisions at small time intervals, usually by simulation.

Midblock stop – A transit stop located at a point away from intersections.

Minimum green – The least amount of time that a green signal indication will be displayed when a signal phase is activated.

Minimum recall – A form of phase recall where the controller places a continuous call for vehicle service on the phase and then services the phase until its minimum green interval times out. The phase can be extended if actuations are received.

Minor movement – A vehicle making a specific directional entry into an unsignalized intersection that must yield to other movements.

Minor street – The street controlled by STOP signs at a two-way STOP-controlled intersection.

Mixed-traffic operation – Operation of a transit mode in lanes shared with other roadway users.

Mobility – The ability of people and goods to travel from place to place.

Mode – See *travel mode.*

Model – A procedure that uses one or more algorithms to produce a set of numerical outputs describing the operation of a segment or system, given a set of numerical inputs.

Model application – The physical configuration and operational conditions to which a traffic analysis tool is applied.

Mountainous terrain – Any combination of grades and horizontal and vertical alignment that causes heavy vehicles to operate at crawl speed for significant distances or at frequent intervals.

Move-up time – The time it takes a vehicle to move from second position into first position on an approach to an all-way STOP-controlled intersection.

Movement capacity – The capacity of a specific traffic stream at a STOP-controlled intersection approach, assuming that the traffic has exclusive use of a separate lane.

Movement groups – An organization of traffic movements at a signalized intersection to facilitate data entry. A separate movement group is established for (*a*) each turn movement with one or more exclusive turn lanes and (*b*) the through movement (inclusive of any turn movements that share a lane).

Multilane highway – A highway with at least two lanes for the exclusive use of traffic in each direction, with no control or partial control of access, but that may have periodic interruptions to flow at signalized intersections no closer than 2 mi.

Multilane roundabout – A roundabout with more than one lane on at least one entry and at least part of the circulatory roadway.

Multimodal – Being used by more than one travel mode.

Multimodal analysis – A type of HCM analysis where the LOS of each travel mode on a facility is evaluated simultaneously.

Multiple weaving segment – A portion of a freeway where a series of closely spaced merge and diverge areas creates overlapping weaving movements (between different merge–diverge pairs).

N

Near-side stop – A transit stop located on the approach side of an intersection. Transit vehicles stop to serve passengers before crossing the intersection.

No-passing zone – A segment of a two-lane, two-way highway along which passing is prohibited in one or both directions.

Node – The endpoint of a link. See also *point.*

Nonlocal transit service – Transit service on routes with longer stop spacing than local service (e.g., limited-stop, bus rapid transit, or express routes).

Nonweaving flow – The traffic movements in a weaving segment that are not engaged in weaving movements.

Nonweaving movement – A traffic flow within a weaving segment that does not need to cross paths with another traffic flow while traversing the segment.

Normative model – A mathematical model that identifies a set of parameters providing the best system performance.

O

Off-line bus stop – See *bus stop.*

Off-ramp – A ramp that accommodates diverging maneuvers.

Off-street path – See *shared pedestrian–bicycle path.*

Offset – The time that the reference phase begins (or ends) relative to the system master time zero.

On-line bus stop – See *bus stop.*

On-ramp – A ramp that accommodates merging maneuvers.

On-street transitway – A portion of a street right-of-way dedicated to the transit mode, physically segregated from other traffic, and located in the median or adjacent to one side of the street.

One-sided weaving segment – A weaving segment in which no weaving maneuvers require more than two lane changes to be completed successfully.

Operations-level analysis – An application of an HCM methodology where the user supplies all or nearly all required inputs to the procedure.

Opposing approach – At an all-way STOP-controlled intersection, the approach approximately 180 degrees opposite the subject approach.

Opposing flow rate – The flow rate for the direction of travel opposite to the direction under analysis.

Outputs – The performance measures produced by a model.

Overflow queue – Queued vehicles left over after a green phase at a signalized intersection.

Oversaturated flow – Traffic flow where (*a*) the arrival flow rate exceeds the capacity of a point or segment, (*b*) a queue created from a prior breakdown of a facility has not yet dissipated, or (*c*) traffic flow is affected by downstream conditions.

P

Partial cloverleaf interchange (parclo) – An interchange with one to three (typically two) loop ramps and two to four diagonal ramps, with major turning movements desirably being made by right-turn exits and entrances.

Partial diamond interchange – A diamond interchange with fewer than four ramps, so that not all of the freeway–street or street–freeway movements are served.

Partial stop – A situation where a vehicle slows as it approaches the back of a queue but does not come to a full stop.

Passage time – The maximum amount of time one vehicle actuation can extend the green interval while green is displayed. It is input for each actuated signal phase; also referred to as vehicle interval, extension interval, extension, or unit extension.

Passenger-car equivalent – The number of passenger cars that will result in the same operational conditions as a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions.

Passenger load factor – See *load factor.*

Passenger trip length – The average distance traveled by a passenger on board a transit vehicle.

Passing lane – A lane added to improve passing opportunities in one direction of travel on a conventional two-lane highway.

Passing sight distance – The length of highway required to complete normal passing maneuvers in which the passing driver can determine that there are no potentially conflicting vehicles ahead before beginning the maneuver.

Passive priority – A form of traffic signal priority that is pretimed, such as the setting of a street's signal progression to favor buses.

Pavement condition rating – A description of the road surface in terms of ride quality and surface defects.

Peak hour – The hour of the day in which the maximum volume occurs. See also *analysis hour.*

Peak hour factor (PHF) – The hourly volume during the analysis hour divided by the peak 15-min flow rate within the analysis hour; a measure of traffic demand fluctuation within the analysis hour.

Pedestrian – An individual traveling on foot.

Pedestrian clear interval – Time provided for pedestrians who depart the curb during the WALK indication to reach the opposite curb (or the median).

Pedestrian crosswalk – A connection between pedestrian facilities across sections of roadway used by automobiles, bicycles, and transit vehicles. Crosswalks can be marked or unmarked.

Pedestrian density – The number of pedestrians per unit of area within a walkway or queuing area.

Pedestrian flow rate – The number of pedestrians passing a point per unit of time. See also *unit width flow rate.*

Pedestrian mode – A travel mode under which a journey (or part of a journey) is made on foot along a roadway or pedestrian facility.

Pedestrian overpass – A grade-separated pedestrian facility over such barriers as wide or high-speed roadways, railroad tracks, busways, or topographic features.

Pedestrian plaza – A large, paved area that serves multiple functions, including pedestrian circulation, special events, and seating.

Pedestrian queuing area – A place where pedestrians stand temporarily while waiting to be served, such as at the corner of a signalized intersection.

Pedestrian recall – A form of phase recall where the controller places a continuous call for pedestrian service on the phase and then services the phase for at least an amount of time equal to its walk and pedestrian clear intervals (longer if vehicle detections are received).

Pedestrian service time – The elapsed time starting with the first pedestrian's departure from the corner to the last pedestrian's arrival at the far side of the crosswalk.

Pedestrian space – The average area provided for pedestrians in a moving pedestrian stream or pedestrian queue.

Pedestrian start-up time – The time for a platoon of pedestrians to get under way following the beginning of the walk interval.

Pedestrian street – A street dedicated to pedestrian use on a full- or part-time basis.

Pedestrian underpass – A grade-separated pedestrian facility under such barriers as wide or high-speed roadways, railroad tracks, busways, or topographic features.

Pedestrian walkway – A pedestrian facility similar to a sidewalk in construction but located well away from the influence of automobile traffic.

Percent free-flow speed – The average travel speed divided by the free-flow speed.

Percent time-spent-following – The average percent of total travel time that vehicles must travel in platoons behind slower vehicles because of inability to pass on a two-lane highway.

Performance measure – A quantitative or qualitative characterization of some aspect of the service provided to a specific road user group.

Permitted plus protected – Compound left-turn protection that displays the permitted phase before the protected phase.

Permitted turn – A left or right turn at a signalized intersection that is made by a vehicle during a time in the cycle in which the vehicle does not have the right-of-way.

Person capacity – The maximum number of persons who can pass a given point during a specified period under prevailing conditions.

Phase – The part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals. A phase includes the green, yellow change, and red clearance intervals.

Phase pair – See *barrier pair.*

Phase pattern – The alternation of right-of-way among various traffic streams at an all-way STOP-controlled intersection.

Phase recall – A call made by the controller for a specified phase each time the controller is servicing a conflicting phase.

Phase sequence – 1. The sequence of service provided to each traffic movement. **2.** A description of the order in which the left-turn movements are served relative to the through movements.

Planning-level analysis – An application of the HCM generally directed toward broad issues such as initial problem identification (e.g., screening a large number of locations for potential operations deficiencies), long-range analyses, and statewide performance monitoring. Nearly all inputs to the analysis may be defaulted.

Platoon – A group of vehicles or pedestrians traveling together as a group, either voluntarily or involuntarily because of signal control, geometrics, or other factors.

Platoon ratio – A description of the quality of signal progression computed as the demand flow rate during the green indication divided by the average demand flow rate.

Point – A place along a facility where (*a*) conflicting traffic streams cross, merge, or diverge; (*b*) a single traffic stream is regulated by a traffic control device; or (*c*) there is a significant change in the segment capacity (e.g., lane drop, lane addition, narrow bridge, significant upgrade, start or end of a ramp influence area).

Potential capacity – The capacity of a specific movement at a STOP-controlled intersection approach, assuming that it is unimpeded by pedestrian or higher-rank movements and has exclusive use of a separate lane.

Precision – The size of the estimation range for a measured quantity.

Preliminary engineering–level analysis – An HCM application conducted to support planning decisions related to roadway design concept and scope, when alternatives analyses are performed, or to assess proposed systemwide policies. Many of the inputs to the analysis will be defaulted.

Prepositioning – A deliberate driver choice of one lane over another at an intersection in anticipation of a turn at a downstream intersection.

Pretimed control – A signal control in which the cycle length, phase plan, and phase times are preset to repeat continuously.

Prevailing condition – The geometric, traffic, control, and environmental conditions during the analysis period.

Progression – The act of various controllers providing specific green indications in accordance with a time schedule to permit continuous operation of groups of vehicles along the street at a planned speed.

Protected plus permitted – Compound left-turn protection at a signalized intersection that displays the protected phase before the permitted phase.

Protected turn – The left or right turns at a signalized intersection that are made by a vehicle during a time in the cycle when the vehicle has the right-of-way.

Q

Quality of service – A description of how well a transportation facility or service operates from a traveler's perspective.

Quantity of service – A measure of the utilization of a transportation system.

Queue – A line of vehicles, bicycles, or persons waiting to be served due to traffic control, a bottleneck, or other causes.

Queue delay – 1. The amount of time that a vehicle spends in a queued state. **2.** When computed from vehicle trajectories, it is the accumulated time step delay over all time steps in which the vehicle is in a queue.

Queue discharge flow – A traffic flow that has passed through a bottleneck and, in the absence of another downstream bottleneck, is accelerating to the free-flow speed of the freeway.

Queue jump – A short bus lane section (often shared with a right-turn lane), in combination with an advance green indication for the lane, that allows buses to move past a queue of cars at a signal.

Queue length – The distance between the upstream and downstream ends of the queue.

Queue spillback – A condition where the back of a queue extends beyond the available storage length, resulting in potential interference with upstream traffic movements.

Queue storage ratio – The maximum back of queue as a proportion of the available storage on the subject lane or link.

Queued state – A condition when a vehicle is within one car length (20 ft) of a stopped vehicle and is itself in a stopped state (i.e., has slowed to less than 5 mi/h).

R

Ramp – A dedicated roadway providing a connection between two other roadways; at least one of the roadways a ramp connects is typically a high-speed facility such as a freeway, multilane highway, or C-D roadway.

Ramp–freeway junction – The point of connection between a ramp and a high-speed facility such as a freeway, multilane highway, or C-D roadway.

Ramp meter – A traffic signal that controls the entry of vehicles from a ramp onto a limited-access facility; the signal allows one or two vehicles to enter on each green or green flash.

Ramp roadway – See *ramp.*

Ramp–street junction – See *interchange ramp terminal.*

Rank – The hierarchy of right-of-way among conflicting traffic streams at a two-way STOP-controlled intersection.

Reasonable expectancy – The concept that the stated capacity for a given system element is one that can be achieved repeatedly during peak periods, rather than being the absolute maximum flow rate that could be observed.

Receiving lanes – Lanes departing an intersection.

Recreational vehicle – A heavy vehicle, generally operated by a private motorist, for transporting recreational equipment or facilities; examples include campers, motor homes, and vehicles towing boat trailers.

Red clearance interval – A brief period of time following the yellow indication during which the signal heads associated with the ending phase and all conflicting phases display a red indication.

Red time – The period in the signal cycle during which, for a given phase or lane group, the signal is red.

Reentry delay – Delay experienced by buses leaving a bus stop, when they must wait for a gap in traffic before reentering the travel lane.

Reference phase – One of the two coordinated phases (i.e., Phase 2 or 6).

Regression model – A model that uses field or simulated data to develop statistically derived relationships between particular model inputs and performance measures such as capacity and delay.

Residual queue – The unmet demand at the end of an analysis period resulting from operation while demand exceeded capacity.

Rest-in-walk mode – A signal controller setting in which the phase will dwell in walk as long as there are no conflicting calls. When a conflicting call is received, the pedestrian clear interval will time to its setting value before ending the phase.

Restrictive median – A median (for example, a raised curb) that prevents or discourages vehicles from crossing the opposing traffic lanes.

Right-turn-on-red – The ability to make a right turn at a signalized intersection when a red indication is displayed, after stopping and only when no conflicting vehicular or pedestrian traffic is present.

Ring – A set of phases operating in sequence.

Roadside obstruction – An object or barrier along a roadside or median that affects traffic flow, whether continuous (e.g., a retaining wall) or not continuous (e.g., light supports or bridge abutments).

Roadway – That portion of a highway improved, designed, or ordinarily used for vehicular travel and parking lanes but exclusive of the sidewalk, berm, or shoulder even though such sidewalk, berm, or shoulder is used by persons riding bicycles or other human-powered vehicles.

Roadway characteristic – A geometric characteristic of a street or highway, including the type of facility, number and width of lanes (by direction), shoulder widths and lateral clearances, design speed, and horizontal and vertical alignments.

Rolling terrain – Any combination of grades and horizontal or vertical alignment that causes heavy vehicles to reduce their speed substantially below that of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length of time or at frequent intervals.

Roundabout – An intersection with a generally circular shape, characterized by yield on entry and circulation around a central island.

Rubbernecking – The slowing of motorists to observe a traffic incident.

Running speed – See *average running speed.*

Running time – The time that a vehicle traverses a length of roadway excluding any delay related to a control device.

Rural – An area with widely scattered development and a low density of housing and employment.

S

Saturation flow rate – The equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced.

Saturation headway – 1. At a signalized intersection, the average headway between vehicles occurring after the fourth vehicle in the queue and continuing until the last vehicle in the initial queue clears the intersection. **2.** At an all-way STOP-controlled intersection, the time between departures of successive vehicles on a given approach for a

particular case, assuming a continuous queue.

Scenario – See *model application.*

Segment – 1. For interrupted flow facilities, a link and its boundary point(s). **2.** For uninterrupted flow facilities, a portion of a facility between two points.

Segment delay – 1. The delay experienced by a vehicle since it left the upstream node (usually another signal), including traffic delay, incident delay, control delay, and geometric delay. **2.** When calculated from vehicle trajectories, the time actually taken to traverse a segment minus the time it would have taken to traverse the segment at the target speed. The segment delay on any time step is equal to the time step delay; segment delays accumulated over all time steps in which a vehicle is present on the segment represent the segment delay for that vehicle.

Segment initialization – The process of determining the appropriate number of vehicles in each segment as a precursor to estimating the number of vehicles on each freeway segment for each time step under oversaturated conditions.

Semiactuated control – A signal control in which some approaches (typically on the minor street) have detectors and some of the approaches (typically on the major street) have no detectors.

Sensitivity analysis – A technique for exploring how model outputs change in response to changes in model inputs, implemented by varying one input at a time over its reasonable range while holding all other inputs constant.

Service flow rate – The maximum directional rate of flow that can be sustained in a given segment under prevailing roadway, traffic, and control conditions without violating the criteria for LOS *i*.

Service measure – A performance measure used to define LOS for a transportation system element.

Service time – At an all-way STOP-controlled intersection, the departure headway minus the move-up time.

Service volume – The maximum hourly directional volume that can be sustained in a given segment without violating the criteria for LOS *i* during the worst 15 min of the hour (period with the highest density) under prevailing roadway, traffic, and control conditions.

Shared lane – 1. A lane shared by more than one movement. **2.** A bicycle facility where bicycles share a travel lane with motorized vehicular traffic.

Shared-lane capacity – The capacity of a lane at an intersection that is shared by two or three movements.

Shared pedestrian–bicycle path – A path physically separated from highway traffic for the use of pedestrians, bicyclists, runners, inline skaters, and other nonmotorized users.

Shock wave – A change or discontinuity in traffic conditions. For example, a shock wave is generated when the signal turns red, and it moves upstream as vehicles arriving at the queue slow down. A shock wave is also generated when the signal turns green, and it moves downstream as the first set of vehicles discharge from the signal.

Short length – The distance within a weaving segment over which lane changing is not prohibited or dissuaded by markings.

Shoulder – A portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles; emergency use; and lateral support of the subbase, base, and surface courses.

Shoulder bikeway – A bicycle facility where bicyclists use a paved shoulder, separated by striping from motor vehicle traffic, for travel along a roadway.

Shoulder bypass lane – A portion of the paved shoulder opposite the minor-road leg at a three-leg intersection, marked as a lane for through traffic to bypass vehicles that are slowing or stopped to make a left turn.

Shy distance – The buffer that pedestrians give themselves to avoid accidentally stepping off the curb, brushing against a building face, or getting too close to pedestrians standing under awnings or window shopping.

Side street – See *minor street.*

Sidepath – A shared pedestrian–bicycle path located parallel and in proximity to a roadway.

Sidewalk – A pedestrian facility located parallel and in proximity to a roadway.

Simple weaving segment – A weaving segment formed by a single merge point followed by a single diverge point.

Simultaneous gap out – A controller mode requiring that both phases reach a point of being committed to terminate (via gap out, max out, or force-off) at the same time.

Single entry – A mode of operation (in a multiring controller) in which a phase in one ring can be selected and timed alone if there is no demand for service in a nonconflicting phase on the parallel ring(s).

Single-point urban interchange – A diamond interchange that combines all the ramp movements into a single signalized intersection.

Single-stage gap acceptance – A condition where no median refuge area is available for minor-street drivers to store in, so that minor-street drivers must evaluate gaps in both major-street directions simultaneously.

Sketch-planning tools – Tools that produce general order-of-magnitude estimates of travel demand and transportation system performance under different transportation system improvement alternatives.

Soft conversion – The conversion of a value from U.S. customary units to metric units (or vice versa) through the application of a conversion factor, such as multiplying 12 ft by 0.305 ft/m, resulting in 3.66 m. See also *hard conversion.*

Space – See *pedestrian space.*

Space gap – See *gap.*

Space mean speed – An average speed based on the average travel time of vehicles to traverse a length of roadway.

Spacing – The distance between two successive vehicles in a traffic lane, measured from the same common feature of the vehicles (e.g., rear axle, front axle, or front bumper).

Spatial stop rate – The ratio of stop rate to facility length.

Spatial variability – Variability in measured values, such as the percentage of trucks in the traffic stream, from one location to another within an area or from one area to another.

Special events – Sources of high demand that occur at known times relatively infrequently, resulting in traffic flow patterns that vary substantially from the typical situation.

Specific grade – A single grade of a roadway segment or extended roadway segment expressed as a percentage.

Speed – A rate of motion expressed as distance per unit of time.

Speed harmonization – A technique to reduce the shock waves that typically occur when traffic abruptly slows upstream of a bottleneck or for an incident, through the use of variable speed limits or advisory speed signs.

Spillback – See *queue spillback.*

Spillover – A condition occurring when pedestrians begin to use more than the provided sidewalk or walkway space (e.g., by stepping into the street) to travel at their desired speed.

Split – The segment of the cycle length allocated to each phase or interval that may occur. In an actuated controller unit, split is the time in the cycle allocated to a phase—the sum of the green, yellow change, and red clearance intervals for a phase.

Split-diamond interchange – Diamond interchanges in which freeway entry and exit ramps are separated at the street level, creating four intersections.

Stairway – A pedestrian facility that ascends a grade via a series of steps and landings.

Standee – A passenger standing in a transit vehicle.

Start-up lost time – The additional time consumed by the first few vehicles in a queue at a signalized intersection above and beyond the saturation headway because of the need to react to the initiation of the green phase and to accelerate.

Static flow model – A mathematical model in which the traffic flow rate and origin–destination volumes are constant.

Stochastic model – A mathematical model that uses random number generation for the determination of at least one parameter.

Stop rate – The count of full stops divided by the number of vehicles served.

Stop spacing – See *average bus stop spacing.*

Stopped delay – The amount of time that a vehicle is stopped. When calculated from vehicle trajectories, it is equal to the time step delay on any step in which the vehicle is in a stopped state. Time step delays accumulated over all time steps in which the vehicle was in the stopped state represent the stopped delay for that vehicle.

Stopped state – A condition when a vehicle is traveling at less than 5 mi/h.

Storage length – The length of turn lane available for storing queued vehicles.

Street – See *highway.*

Street corner – The area encompassed within the intersection of two sidewalks.

Streetcar mode – A transit mode operated by vehicles that receive power from overhead wires and run on tracks. Compared with light rail, streetcars are generally shorter and narrower, are more likely to have on-board fare collection, make more frequent stops, and are more likely to operate in mixed traffic.

Study period – See *analysis period*.

Subject approach – The approach under study at two-way and all-way STOP-controlled intersections.

Suburban street – A street with low-density driveway access on the periphery of an urban area.

Sustained spillback – A result of oversaturation, where a queue does not dissipate at the end of each cycle but remains present until the downstream capacity is increased or the upstream demand is reduced.

System – All the transportation facilities and modes within a particular region.

System elements – Components of a transportation system, including points, segments, facilities, corridors, and areas.

T

Taper area – An area characterized by a reduction or increase in pavement width to direct traffic.

Target speed – In a simulation tool, the speed at which a driver would prefer to travel; it differs from the free-flow speed in that most simulation tools apply a "driver aggressiveness" factor to the free-flow speed to determine a target speed.

Temporal variability – Variability in measured values, such as hourly traffic volumes, that occurs from day to day or month to month at a given location.

Terrain – See *general terrain.*

Three-level diamond interchange – A diamond interchange with two divided levels so that both facilities provide continuous through movements.

Through vehicles – All vehicles passing directly through a street segment and not turning.

Tight urban diamond interchange – A diamond interchange with a separation of less than 400 ft between the two intersections.

Time interval – See *analysis period.*

Time interval scale factor – The ratio of the total facility entrance counts to total facility exit counts.

Time mean speed – The average speed of vehicles observed passing a point on a highway.

Time gap – See *gap.*

Time–space domain – A specification of the freeway sections included in the defined facility and an identification of the time intervals for which the analysis is to be conducted.

Time step delay – The length of a time step minus the time it would have taken a vehicle to cover the distance traveled in the step at the target speed.

Time-varying flow model – A simulation model in which flow changes with time.

Tool – See *traffic analysis tool.*

Total lateral clearance (TLC) – The sum of the right-side and left-side lateral clearances along a multilane highway.

Total lost time – See *lost time.*

Total ramp density – The average number of on-ramp, off-ramp, major merge, and major diverge junctions per mile. It applies to a 6-mi segment of freeway facility, 3 mi upstream and 3 mi downstream of the midpoint of the study segment.

Traffic analysis tool – A software product used for traffic analysis that includes, at a minimum, a computational engine and a user interface.

Traffic condition – A characteristic of traffic flow, including distribution of vehicle types in the traffic stream, directional distribution of traffic, lane use distribution of traffic, and type of driver population on a given facility.

Traffic control device – A sign, signal, marking, or other device used to regulate, warn, or guide traffic.

Traffic delay – The component of delay that results when the interaction of vehicles causes drivers to reduce speed below the free-flow speed.

Traffic incidents – Occurrences, such as crashes, stalled cars, and debris in the roadway, that do not occur every day. These incidents reduce a roadway's capacity and create variation in day-to-day travel times along the roadway.

Traffic pressure – The display of aggressive driving behavior for a large number of drivers during high-demand traffic

conditions. Under such conditions, a large number of drivers accept shorter headways during queue discharge than they would under different circumstances.

Traffic signal delay – Delay experienced by a bus that arrives at a near-side stop during the green interval, serves its passengers during portions of the green and red intervals, and then must wait for the traffic signal to turn green again before proceeding. See also *control delay.*

Traffic signal optimization tool – A tool primarily designed to develop optimal signal phasing and timing plans for isolated signalized intersections, arterial streets, or signal networks.

Transit frequency – The count of scheduled fixed-route transit vehicles that stop on or near an urban street segment during the analysis period.

Transit mode – A travel mode in which vehicles (including buses, streetcars, and street-running light rail) stop at regular intervals along the roadway to pick up and drop off passengers.

Transit reliability – A measure of the time performance and the regularity of headways between successive transit vehicles affecting the amount of time passengers must wait at a transit stop as well as the consistency of a passenger's arrival time at a destination.

Transit signal preemption – The transfer of normal operation of a traffic signal to a special control mode serving a transit vehicle.

Transit signal priority – Adjustments to traffic signal timing to provide more usable green time to transit vehicles. See also *active priority* and *passive priority.*

Transitway – See *on-street transitway.*

Travel demand models – Models that forecast long-term future travel demand on the basis of current conditions and projections of socioeconomic characteristics and changes in transportation system design.

Travel mode – 1. A transport category characterized by specific right-of-way, technological, and operational features. **2.** A particular form of travel, for example, walking, bicycling, traveling by automobile, or traveling by bus.

Travel speed – See *average travel speed.*

Travel time – The average time spent by vehicles traversing a highway segment, including control delay.

Travel time rate – The reciprocal of speed, expressed as time per unit distance traveled.

Travel time reliability – 1. The probability of "on-time" arrival (i.e., the probability that a trip is completed below a certain threshold time). **2.** The variability in travel time for a given trip due to unforeseen causes such as variations in demand or an incident.

Traveler information systems – An integration of technologies that allow the general public to access real-time or near real-time data on traffic factors such as incident conditions, travel time, and speed.

Traveler perception model – A model that estimates the average response or range of responses that travelers would give to a given set of conditions (typically operational or design in nature).

Truck – A heavy vehicle engaged primarily in the transport of goods and materials or in the delivery of services other than public transportation.

Turn lane – See *exclusive turn lane.*

Turnout – A short segment of a lane—usually a widened, unobstructed shoulder area—added to a two-lane, two-way highway, allowing slow-moving vehicles to leave the main roadway and stop so that faster vehicles can pass.

Two-lane highway – A roadway with a two-lane cross section, one lane for each direction of flow, on which passing maneuvers must be made in the opposing lane.

Two-phase pattern – A type of operation at an all-way STOP-controlled intersection where drivers from opposing approaches enter the intersection at roughly the same time.

Two-sided weaving segment – A weaving segment in which at least one weaving maneuver requires three or more lane changes to be completed successfully or in which a single-lane on-ramp is closely followed by a single-lane off-ramp on the opposite side of the freeway.

Two-stage gap acceptance – A condition where a median refuge area is available for minor-street drivers so that drivers sequentially evaluate and use gaps in the near-side major-street traffic stream, followed by gaps in the far-side major-street traffic stream.

Two-way left-turn lane – A lane in the median area that extends continuously along a street or highway and is marked to provide a deceleration and storage area, out of the

through-traffic stream, for vehicles traveling in either direction to use in making left turns at intersections and driveways.

Two-way STOP-controlled – The type of traffic control at an intersection where drivers on the minor street or drivers turning left from the major street wait for a gap in the major-street traffic to complete a maneuver.

U

Uncertainty – The range within which a model's estimate of a value is statistically likely to vary from the actual value.

Uncontrolled – Lacking a traffic control device that interrupts traffic flow (e.g., a traffic signal, STOP sign, or YIELD sign).

Undersaturated flow – Traffic flow where (*a*) the arrival flow rate is lower than the capacity of a point or segment, (*b*) no residual queue remains from a prior breakdown of the facility, and (*c*) traffic flow is unaffected by downstream conditions.

Uniform delay – The first term of the equation for lane group control delay, assuming constant arrival and departure rates during a given time period.

Uninterrupted flow – Traffic flow on facilities that have no fixed causes of delay or interruption external to the traffic stream; examples include freeways and unsignalized sections of multilane and two-lane rural highways.

Unit extension – See *passage time.*

Unit width flow rate – The pedestrian flow rate expressed as pedestrians per minute per unit of walkway or crosswalk width.

Unmet demand – The number of vehicles on a signalized lane group that have not been served at any point in time as a result of operation in which demand exceeds capacity in either the current or previous analysis period. This does not include the normal cyclical queue formation on the red and discharge on the green phase. See also *initial queue* and *residual queue.*

Unsignalized intersection – An intersection not controlled by traffic signals.

Upstream – The direction from which traffic is flowing.

Urban – An area typified by high densities of development or concentrations of population, drawing people from several areas within a region.

Urban street – A street with relatively high density of driveway access located in an urban area and with traffic signals or interrupting STOP or YIELD signs no farther than 2 mi apart.

Urban street facilities – Extended sections of collector or arterial streets that include the impacts of traffic signals or other traffic control along the street.

Urban street segment – A length of urban street from one boundary intersection to the next, including the upstream boundary intersection but not the downstream boundary intersection.

User perception variability – Variability in user responses that occurs when different users experiencing identical conditions are asked to rate the conditions.

Utility – A measure of the value a traveler places on a trip choice.

V

Validation – The process by which the analyst checks the overall model-predicted traffic performance for a street–road system against field measurements of traffic performance that were not used in the calibration process.

Variability – The day-to-day variation in other dimensions of congestion within a transportation system.

Vehicle capacity – The maximum number of vehicles that can pass a given point during a specified period under prevailing roadway, traffic, and control conditions.

Vehicle trajectory analysis – The development of performance measures from the properties of time–space trajectories of individual vehicles.

Verification – The process by which a software developer and other researchers check the accuracy of a software implementation of traffic operations theory.

Volume – The total number of vehicles or other roadway users that pass over a given point or section of a lane or roadway during a given time interval, often 1 h.

Volume-to-capacity (*v/c*) ratio – The ratio of flow rate to capacity for a system element.

W

Walk interval – A period of time intended to give pedestrians adequate time to perceive the WALK

indication and depart the curb before the pedestrian clear interval begins.

Walkway – See *pedestrian walkway*.

Wave speed – The speed at which a shock wave travels upstream or downstream through traffic.

Weaving – The crossing of two or more traffic streams traveling in the same direction along a significant length of highway, without the aid of traffic control devices (except for guide signs).

Weaving configuration – The organization and continuity of lanes in a weaving segment, which determines lane-changing characteristics.

Weaving flow – The traffic movements in a weaving segment that are engaged in weaving movements.

Weaving length – See *base length, maximum weaving length,* and *short length.*

Weaving movement – A traffic flow within a weaving segment (on-ramp to mainline or mainline to off-ramp) that must cross paths with another traffic flow while traversing the segment.

Weaving segment – See *freeway weaving segment*.

Weaving segment influence area – The base length of the weaving segment plus 500 ft upstream of the entry point to the weaving segment and 500 ft downstream of the exit point from the weaving segment; entry and exit points are defined as the points where the appropriate edges of the merging and diverging lanes meet.

Weaving width – The total number of lanes between the entry and exit gore areas within a weaving segment, including the auxiliary lane, if present.

Work zone – A segment of highway in which maintenance or construction operations reduce the number of lanes available to traffic or affect the operational characteristics of traffic flowing through the segment.

Y

Yellow change interval – The period of time that a yellow indication is displayed to alert drivers to the impending presentation of a red indication.

2. LIST OF SYMBOLS

This section lists and defines the symbols and abbreviations used in the HCM, along with their units if applicable. If a symbol has more than one meaning, the chapter or chapters of the specific use are cited in parentheses after the definition.

$\%HV$	percentage of heavy vehicles (decimal)
$\%OHP$	percentage of segment with occupied on-highway parking
$\%VL_i$	percentage of traffic present in lane L_i
$\%VL_{max}$	percentage of the total approach flow in the lane with the highest volume (decimal)
A	critical flow ratio for the arterial movements
a_1	passenger load weighting factor
$AADT$	annual average daily traffic (veh/day)
A_i	expected passings per minute of mode i by average bicycle
A_p	pedestrian space (ft²/p)
A_T	expected active passings per minute by the average bicycle during the peak 15 min
ATS_d	average travel speed in the analysis direction (mi/h)
ATS_F	average travel speed for the facility (mi/h)
ATS_i	average travel speed for directional segment i (mi/h)
ATS_{pl}	average travel speed in the analysis segment as affected by a passing lane (mi/h)
AVO_i	average vehicle occupancy on segment i (p/veh)
$b_{d,j}$	destination adjustment factor j
$BFFS$	base free-flow speed (mi/h)
b_i	bunching factor for lane group i
$BLOS$	bicycle level of service score
$b_{o,i}$	origin adjustment factor i
$BPTSF_d$	base percent time-spent-following in the analysis direction
c	base capacity
C	cycle length (s)
$c_{i,PCE}$	capacity for lane i (pc/h)
C'	cycle length (steps)
c_a	adjusted mainline capacity (veh/h)
c_A	average capacity (veh/h)
$c_{a,l,e,p}$	available capacity of an exclusive-lane lane group with protected left-turn operation (veh/h)
c'_a	adjusted capacity of work or construction zone
c_{act}	actual capacity of the flared lane (veh/h)
c_b	capacity of the bicycle lane (bicycles/h)
$c_{\text{bypass},pce}$	capacity of the bypass lane, adjusted for heavy vehicles (pc/h)
c_{dATS}	capacity in the analysis direction under prevailing conditions based on ATS (pc/h)
c_{dPTSF}	capacity in the analysis direction under prevailing conditions based on PTSF (pc/h)

C_e	equilibrium cycle length (s)
$c_{e,L,pce}$	capacity of the left entry lane, adjusted for heavy vehicles (pc/h)
$c_{e,R,pce}$	capacity of the right entry lane, adjusted for heavy vehicles (pc/h)
CG_{DS}	common green time with demand starvation potential (s)
CG_{RD}	common green time between the upstream ramp green and the downstream arterial through green (s)
CG_{UD}	common green time between the upstream through green and downstream through green (s)
ci	set of critical phases on the critical path
c_i	capacity for lane i (veh/h)
c_{IFL}	capacity of a basic freeway segment with the same free-flow speed as the weaving segment under equivalent ideal conditions, per lane (pc/h/ln)
c_{IW}	capacity of all lanes in the weaving segment under ideal conditions (pc/h)
c_{IWL}	capacity of the weaving segment under equivalent ideal conditions (pc/h/ln)
c_l	capacity of a left-turn movement with permitted left-turn operation (veh/h)
$c_{l,e}$	capacity of an exclusive-lane lane group with permitted left-turn operation (veh/h)
$c_{l,e,p}$	capacity of an exclusive-lane lane group with protected left-turn operation (veh/h)
c_{L+TH}	capacity of the through and left-turn movements (veh/h)
$c_{m,j}$	capacity of movement j
$c_{m,x}$	capacity of movement x (veh/h)
$c_{m,y}$	movement capacity of the y movement in the subject shared lane (veh/h)
C_{max}	maximum cycle length (s)
c_{mg}	merge capacity (veh/h)
C_{min}	minimum cycle length (s)
c_{nm}	nonmerge capacity for the inside lane (veh/h)
$c_{p,x}$	potential capacity of movement x (veh/h)
c_{pce}	lane capacity adjusted for heavy vehicles (pc/h)
CP_i	change period (yellow change interval plus red clearance interval) for phase i (s)
$c_{q\mid r}$	shared lane capacity for upstream right-turn traffic movement (veh/h)
c_R	capacity of the right-turn movement (veh/h)
$c_{r,x}$	capacity of movement x assuming random flow during the unblocked period
c_s	saturated capacity (veh/h)
CS	critical sum (veh/h)
c_{sep}	capacity of the lane if both storage areas were infinitely long (veh/h)
c_{SH}	capacity of the shared lane (veh/h)
c_{sl}	capacity of a shared-lane lane group with permitted left-turn operation (veh/h)
c_{thru}	capacity for the exiting through movement (veh/h)
c_{turn}	capacity for the exiting turn movement (veh/h)
CV	critical phase flow rate (veh/h)
d	demand flow rates (veh/h, Chapter 10); control delay (s/veh, Chapter 17); interchange delay (s/veh, Chapter 22)
D	density (pc/mi/ln, Chapter 14); distance between the two intersections of the interchange (ft, Chapter 22)
d_1	conditional delay to first through vehicle (s/veh)

d_{1b} baseline uniform delay (s/veh)

d_2 incremental delay (s/veh)

d_3 initial queue delay (s/veh)

d_A control delay on the approach (s/veh)

d_a acceleration/deceleration delay (s)

D_a access-point density on segment (points/mi)

$D_{a,i}$ adjusted volume for destination *j* (veh/h)

$d_{A,j}$ approach control delay for approach *j* (s/veh)

$d_{A,x}$ control delay on approach *x* (s/veh)

d_{ad} transit vehicle acceleration/deceleration delay due to a transit stop (s/veh)

$d_{ap,i}$ delay due to left and right turns from the street into access point intersection *i* (s/veh)

$d_{ap,l}$ through vehicle delay due to left turns (s/veh)

$d_{ap,r}$ through vehicle delay due to right turns (s/veh)

$d_{approach}$ control delay for the approach (s/veh)

d_b bicycle delay (s/bicycle)

D_c distance to nearest signal-controlled crossing (ft)

D_d diversion distance (ft)

DDHV directional design-hour volume (veh/h)

D_F average density for the facility (pc/mi/ln)

d_g average pedestrian gap delay (s)

d_{gd} average gap delay for pedestrians who incur nonzero delay

D_i person-hours of delay on segment *i* (Chapter 2); density for segment *i* (pc/mi/ln, Chapter 10)

d_i vehicle demand on segment *i* (veh, Chapter 2) ; control delay for lane *i* (s/veh, Chapter 19)

d_I intersection control delay (s/veh)

$d_{\text{intersection}}$ control delay for the entire intersection (s/veh)

D_j volume for destination *j* (veh/h)

d_l computed control delay for the left-turn movements (s/veh)

$d_{M,LT}$ delay to major left-turning vehicles (s/veh)

D_{MD} density in the major diverge influence area (which includes all approaching freeway lanes) (pc/mi/ln)

d_{mg} merge delay (s/veh)

d_{nm} nonmerge delay for the inside lane (s/veh)

d_{other} delay due to other sources along the route (s/veh)

D_p phase duration (s)

d_p average pedestrian delay (s)

$D_{p,a}$ phase duration for phase *a*, which occurs just before phase *b* (s)

$D_{p,b}$ phase duration for phase *b*, which occurs just after phase *a* (s)

$d_{p,d}$ pedestrian delay when traversing Crosswalk D (s/p)

$D_{p,l}$ phase duration for left-turn phase *l* (s)

$D_{p,mi}$ duration of the phase serving the minor-street through movement (s)

$D_{p,t}$ phase duration for coordinated phase *t* (s)

d_{pc}	pedestrian delay when crossing the segment at a signalized intersection (s/p)
d_{pd}	pedestrian diversion delay (s/p)
D_{ped}	pedestrian density (p/ft^2)
d_{pp}	pedestrian delay when walking parallel to the segment (s/p)
d_{ps}	transit vehicle delay due to serving passengers (s)
d_{pw}	pedestrian waiting delay (s/p)
d_{px}	crossing delay (s/p)
DQ_A	distance to the downstream queue at the beginning of the upstream arterial green (ft)
DQ_i	distance to the downstream queue at the beginning of the upstream green for approach *i* (ft)
DQ_R	distance to the downstream queue at the beginning of the upstream ramp green (ft)
D_R	density in the ramp influence area (pc/mi/ln)
d_r	computed control delay for the right-turn movements (s/veh)
d_{re}	transit vehicle delay reentering the traffic stream from a transit stop (s/veh)
D_S	speed index for off-ramps
d_s	saturated uniform delay (s/veh)
d_{sep}	control delay for the movement considered as a separate lane
d_{sl}	control delay in shared left-turn and through lane group (s/veh)
d_{sr}	control delay in shared right-turn and through lane group (s/veh)
DSV	daily service volumes
D_{sv}	distance between stored vehicles (= 8 ft)
DSV_i	daily service volume for level of service *i* (veh/day)
d_t	control delay in exclusive through lane group (s/veh)
$d_{t,1}$	average delay to through vehicles in the inside lane (s/veh)
$d_{T,i}$	total delay associated with interval *i* (veh-s)
$d_{t\|r}$	through vehicle delay per right-turn maneuver (s/veh)
d_{ts}	delay due to a transit vehicle stop for passenger pickup (s/stop)
D_{up}	unbalanced phase duration (s)
$D_{up,i}$	unbalanced phase duration for phase *i* (s)
d_{vq}	time-in-queue per vehicle (s/veh)
dx_j	length of discrete segment *j* (mi)
e	ridership elasticity with respect to changes in the travel time rate (Chapter 17); the extension of effective green time into the clearance interval (s, Chapter 22)
E	weighted events per minute
E_L	equivalent number of through cars for a protected left-turning vehicle
$E_{L,m}$	modified through-car equivalent for a protected left-turning vehicle
E_{L1}	equivalent number of through cars for a permitted left-turning vehicle
$E_{L1,m}$	modified through-car equivalent for a permitted left-turning vehicle
E_{L2}	equivalent number of through cars for a permitted left-turning vehicle when opposed by a queue on a single-lane approach
$E_{L2,m}$	modified through-car equivalent for a permitted left-turning vehicle when opposed by a queue on a single-lane approach
e_p	permitted extension of effective green (s)

E_R passenger-car equivalent for recreational vehicles (Chapter 10); equivalent number of through cars for a protected right-turning vehicle (Chapter 18)

$E_{R,ap}$ equivalent number of through cars for a protected right-turning vehicle at an access point

$E_{R,m}$ modified through-car equivalent for a protected right-turning vehicle

E_T passenger-car equivalent for trucks and buses

E_{TC} passenger-car equivalent for trucks operating at crawl speed

F total events on the path (events/h)

f_a adjustment factor for area type (0.90 if CBD, 1.00 otherwise)

f_A adjustment for access points (mi/h)

f_{ad} proportion of transit vehicle stop acceleration/deceleration delay not due to traffic control

f_{bb} adjustment factor for blocking effect of local buses that stop within intersection area

F_{bi} indicator variable for boundary intersection control type (1.0 if signalized, 0.0 if two-way STOP-controlled)

F_{cd} roadway crossing difficulty factor

f_{CS} adjustment for cross section (mi/h)

F_{delay} pedestrian delay adjustment factor

f_{dt} proportion of dwell time occurring during effective green

f_F adjustment for the presence of merge, diverge, and weaving segments along a facility

FFS free-flow speed (mi/h)

f_g adjustment factor for approach grade

$f_{g,ATS}$ grade adjustment factor

$f_{g,PTSF}$ grade adjustment factor for PTSF determination

F_h headway factor

f_{HV} heavy-vehicle adjustment factor

$f_{HV,ATS}$ heavy-vehicle adjustment factor for average travel speed

$f_{HV,e}$ heavy-vehicle adjustment factor for the lane

$f_{HV,PTSF}$ heavy-vehicle adjustment factor for PTSF determination

f_L signal spacing adjustment factor

F_l passenger load factor (passengers/seat)

f_{LC} adjustment for lateral clearance (mi/h)

f_{Lpb} pedestrian adjustment factor for left-turn groups

f_{LS} adjustment for lane and shoulder width (mi/h)

f_{LT} adjustment factor for left-turn vehicle presence in a lane group

f_{LU} adjustment factor for lane utilization

f_{LW} adjustment for lane width (mi/h)

f_M adjustment for median type (mi/h)

F_m number of meeting events (events/h)

$f_{np,ATS}$ adjustment factor for ATS determination for the percentage of no-passing zones in the analysis direction

$f_{np,PTSF}$ adjustment to PTSF for the percentage of no-passing zones in the analysis segment

FO_4 force-off point for Phase 4 (s)

f_p driver population factor

F_p	number of passing events (events/h)
F_p	pavement condition adjustment factor
f_{pb}	pedestrian blockage factor or the proportion of time that one lane on an approach is blocked during 1 h
f_{ped}	entry capacity adjustment factor for pedestrians
$f_{pl,ATS}$	adjustment factor for the effect of passing lane on average travel speed
$f_{pl,PTSF}$	adjustment factor for the impact of a passing lane on percent time-spent-following
f_{Rpb}	pedestrian–bicycle adjustment factor for right-turn groups
f_{RT}	adjustment for right-turning vehicle presence in the lane group
F_S	motorized vehicle speed adjustment factor
f_{sw}	sidewalk width coefficient
f_{TISi}	time-interval scale factor for time period *i*
F_{tt}	perceived travel time factor
f_v	adjustment factor for traffic pressure
F_v	motorized vehicle volume adjustment factor
f_w	adjustment factor for lane width
F_w	cross-section adjustment factor
g	effective green time (s)
G	percent grade (Chapter 17); green time (s, Chapter 22)
$G_{\mid ped,call}$	average green interval given that the phase is called by a pedestrian detection (s)
$G_{\mid veh,call}$	average green interval given that the phase is called by a vehicle detection (s)
g'	effective green time adjusted because of the presence of a downstream queue (s)
G_3	green interval duration for Phase 3 (s)
g_a	available effective green time (s)
GA	green interval for the external arterial approach (s)
g_b	effective green time for the bicycle lane (s)
GD	green interval for the downstream arterial through movement (s)
g_{diff}	supplemental service time for shared single-lane approaches (s)
g_e	green extension time (s)
g_f	time before the first left-turning vehicle arrives and blocks the shared lane (s)
$g_{f,max}$	maximum time before the first left-turning vehicle arrives and within which there are sufficient through vehicles to depart at saturation (s)
g_i	effective green time for lane group *i* (s)
g_l	effective green time for left-turn phase (s)
G_{max}	maximum green setting (s)
G_{min}	minimum green setting (s)
g_p	effective green time for permitted left-turn operation (s)
G_p	displayed green interval corresponding to g_p (s)
$G_{p,min}$	minimum green interval duration based on pedestrian crossing time (s)
g_{ped}	pedestrian service time (s)
$g_{ped,mi}$	pedestrian service time for the phase serving the minor-street through movement (s)
g_{ps}	queue service time during permitted left-turn operation (s)
g_q	opposing queue service time (= g_s for the opposing movement) (s)

G_q	displayed green interval corresponding to g_q (s)
GR	green interval for the left-turning ramp movement (s)
g_s	queue service time (s)
g_u	duration of permitted left-turn green time that is not blocked by an opposing queue (s)
G_u	unbalanced green interval duration for a phase (s)
GU_i	green interval for the upstream approach *i* (s)
$g_{Walk,mi}$	effective walk time for the phase serving the minor-street movement (s)
h	average headway for each through lane
$h_{1\|\Delta<h<H1}$	average headway of those headways between Δ and H_1 (s/veh)
H_1	maximum headway that the first through vehicle can have and still incur delay (s/veh)
h_{adj}	headway adjustment (s)
h_d	departure headway or average time between departures of successive vehicles on a given approach (s)
$h_{HV,adj}$	headway adjustment for heavy vehicles (s)
h_I	saturation headway for the internal through approach (s)
h_{is}	saturation headway or time between departures of successive vehicles on a given approach for a particular case (case *i*) (s)
$h_{LT,adj}$	headway adjustment for left turns (s)
h_{other}	full stop rate due to other sources (stops/veh)
$h_{RT,adj}$	headway adjustment for right turns (s)
H_{seg}	spatial stop rate for the segment (stops/mi)
i	crossing event index
I	adjustment factor for type, intensity, and proximity of work activity (pc/h/ln, Chapter 10); upstream filtering adjustment factor (Chapter 18)
$I_{a,seg}$	automobile traveler perception score for segment
$I_{b,int}$	bicyclist perception score for intersection
$I_{b,link}$	bicyclist perception score for link
$I_{b,seg}$	bicyclist perception score for segment
$I_{b,sig}$	bicycle perception score for signalized intersection
ID	interchange density; the number of interchanges within ±3 mi of the center of the subject weaving segment divided by 6 (int/mi)
I_{LC}	lane-changing intensity; LC_{ALL}/L_S (lc/ft)
$I_{p,int}$	pedestrian perception score for intersection
$I_{p,link}$	pedestrian perception score for link
$I_{p,seg}$	pedestrian perception score for segment
$I_{p,sig}$	pedestrian perception score for signalized intersection
I_{pk}	indicator variable for on-street parking occupancy (= 0 if $p_{pk} > 0.0$)
I_s	interval between vehicle-in-queue counts (s)
I_{sh}	indicator variable for shared lane (= 1.0 if the subject left turn is served in a shared lane; 0 if the subject left turn is served in an exclusive lane)
I_t	indicator variable = 1.0 when equations are used to evaluate delay due to left turns; 0.00001 when equations are used to evaluate delay due to right turns
$I_{t,seg}$	transit passenger perception score for segment

j time step associated with platoon arrival time t'

k incremental delay factor

K proportion of AADT occurring in the peak hour (decimal)

k_i density of users of mode i (users/mi)

k_{min} minimum incremental delay factor

L cycle lost time (s)

l_1 start-up lost time (s)

$l_{1,p}$ permitted start-up lost time (s)

l_2 clearance lost time (s)

L_a available queue storage distance (ft/ln)

L_A length of acceleration lane (ft)

$L_{a,comb}$ available queue storage distance for the combined movement (ft/ln)

$L_{a,lt}$ available queue storage distance for the left-turn movement (ft/ln)

$L_{a,thru}$ available queue storage distance for the through movement (ft/ln)

$L_{a,turn}$ available queue storage distance for the turn movement (ft/ln)

L_B base length of the segment, measured from the points at which the edges of the travel lanes of the merging and diverging roadways converge (ft)

LC_{ALL} total rate of lane changing of all vehicles within the weaving segment (lc/h)

L_{cc} curb-to-curb crossing distance (ft)

LC_{FR} minimum number of lane changes that a freeway-to-ramp weaving vehicle must make to complete the freeway-to-ramp movement successfully

LC_L left-side lateral clearance (ft)

LC_{MIN} minimum rate of lane changing that must exist for all weaving vehicles to complete their weaving maneuvers successfully (lc/h)

LC_{NW} total rate of lane changing by nonweaving vehicles within the weaving segment (lc/h)

LC_R right-side lateral clearance (ft)

LC_{RF} minimum number of lane changes that a ramp-to-freeway weaving vehicle must make to complete the ramp-to-freeway movement successfully

LC_{RR} minimum number of lane changes that must be made by one ramp-to-ramp vehicle to complete a weaving maneuver

LC_W total rate of lane changing by weaving vehicles within the weaving segment (lc/h)

L_d length of Crosswalk D (ft)

L_D length of deceleration lane (ft)

$L_{D\text{-}A}$ lost time on the external arterial approach due to the presence of downstream queue (s)

L_{DOWN} distance between the subject ramp junction and the adjacent downstream ramp junction (ft)

$L_{D\text{-}R}$ lost time on the external ramp approach due to the presence of downstream queue (s)

L_{DS} additional lost time due to demand starvation (s)

L_{ds} length of the stop line detection zone (ft)

$L_{ds,lt}$ length of the stop line detection zone in the left-turn lanes (ft)

$L_{ds,rt}$ length of the stop line detection zone in the right-turn lanes (ft)

$L_{ds,th}$ length of the stop line detection zone in the through lanes (ft)

$L_{D\text{-}Ui}$ lost time on the upstream approach i due to the presence of a downstream queue (s)

L_{EQ} equilibrium separation distance

L_h average vehicle spacing in stationary queue (ft/veh)

L_{HV} stored heavy-vehicle lane length = 45 (ft)

L_i length of segment i (mi)

L_{pc} stored passenger car lane length = 25 (ft)

L_{pt} average passenger trip length (mi)

l_r length of segment traveled by route ($\leq L$) (ft)

L_s distance between adjacent signalized intersections (ft)

L_S short length of the segment, defined as the distance over which lane changing is not prohibited or dissuaded by markings (ft)

L_t total length of directional segment i (mi)

$l_{t,i}$ phase i lost time (s)

L_{TC} left-turn flow rate per cycle (veh/cycle)

L_{UP} distance between the subject ramp junction and the adjacent upstream ramp junction (ft)

L_v detected length of vehicle (ft)

L_{WI} influence area of the weaving segment (ft)

L_{wMAX} maximum length of a weaving segment (ft)

M pedestrian space (ft²/p)

M_1 meetings per minute of users already on path segment

MAH maximum allowable headway (s/veh)

MAH^* equivalent maximum allowable headway for the phase (s/veh)

MAH_c maximum allowable headway for the concurrent phase that also ends at the barrier (s/veh)

$MAH_{lt,e}$ maximum allowable headway for permitted left-turning vehicles in exclusive lane (s/veh)

$MAH_{lt,e,p}$ maximum allowable headway for protected left-turning vehicles in exclusive lane (s/veh)

$MAH_{lt,s}$ maximum allowable headway for permitted left-turning vehicles in shared lane (s/veh)

$MAH_{lt,s,p}$ maximum allowable headway for protected left-turning vehicles in shared lane (s/veh)

$MAH_{rt,e,p}$ maximum allowable headway for protected right-turning vehicles in exclusive lane (s/veh)

$MAH_{rt,s}$ maximum allowable headway for permitted right-turning vehicles in shared lane (s/veh)

MAH_{th} maximum allowable headway for through vehicles (s/veh)

M_{corner} corner circulation area per pedestrian (ft²/p)

M_{cw} crosswalk circulation area per pedestrian (ft²/p)

m_d set of all automobile movements that cross Crosswalk D

m_i average speed of mode i (mi/h)

m_j number of lane groups on approach j

M_S speed index for on-ramps (merge areas)

M_T total number of expected meetings per minute during the peak 15 min

M_y motorist yield rate (decimal)

n_{15} count of vehicles during the peak 15-min period (veh)

$n_{15,mj}$ count of vehicles traveling on the major street during a 15-min period (veh/ln)

n_{60}	count of vehicles during a 1-h period (veh)
N_{ap}	number of influential access-point approaches along the segment
$N_{ap,o}$	number of access-point approaches on the right side in the opposing direction of travel (points)
$N_{ap,s}$	number of access-point approaches on the right side in the subject direction of travel (points)
N_{Arterial}	number of lanes for the upstream arterial through movement
N_b	bus stopping rate on the subject approach (buses/h)
N_c	total number of pedestrians in the crossing platoon (p)
N_{comb}	number of lanes for the combined movement (ln)
N_d	number of traffic lanes crossed when traversing Crosswalk D (ln)
N_{di}	number of pedestrians arriving at the corner each cycle having crossed the major street (p)
N_{do}	number of pedestrians arriving at the corner each cycle to cross the major street (p)
N_e	number of exclusive lanes in movement group (ln)
N_f	number of fully stopped vehicles (veh/ln)
$N_{f,sl}$	number of fully stopped vehicles in shared left-turn and through lane group (veh/ln)
$N_{f,sr}$	number of fully stopped vehicles in shared right-turn and through lane group (veh/ln)
$N_{f,t}$	number of fully stopped vehicles in exclusive through lane group (veh/ln)
n_g	arrival count during green (veh)
N_g	number of lane groups for which t exceeds 0.0 h
n_i	number of lanes serving phase movement i
N_i	number of lanes in segment i
N_l	number of lanes in exclusive left-turn lane group (ln)
N_{lr}	number of lanes in shared left- and right-turn lane group (ln)
N_{lt}	number of lanes in the left-turn bay (ln)
N_m	parking maneuver rate adjacent to lane group (maneuvers/h)
n_{Max}	length of the storage area such that the approach would operate as separate lanes
N_O	number of outer lanes on the freeway (1 for a six-lane freeway; 2 for an eight-lane freeway)
N_p	spatial distribution of pedestrians (p, Chapter 19); number of partial stops (Chapter 30)
n_{ped}	number of conflicting pedestrians (p/h)
N_{ped}	number of pedestrians crossing during an interval (p)
n_q	maximum number of opposing vehicles that could arrive after g_f and before g_u (veh)
N_{qa}	available queue storage (veh)
$N_{qa,comb}$	available queue storage for the combined movement (veh)
$N_{qx,lt}$	maximum queue storage for the left-turn movement (veh)
$N_{qx,thru}$	maximum queue storage for the through movement (veh)
$N_{qx,turn}$	maximum queue storage for a turn movement (veh)
N_r	number of lanes in exclusive right-turn lane group (ln)
$N_{Ramp\text{-}L}$	number of lanes for the upstream ramp left-turning movement
$N_{rtci,d}$	number of right-turn channelizing islands along Crosswalk D
n_s	number of sneakers per cycle (= 2.0 veh)

N_{sl}	number of lanes in shared left-turn and through lane group (ln)
N_{sr}	number of lanes in shared right-turn and through lane group (ln)
N_t	total number of stops
N_{th}	number of through lanes (shared or exclusive) (ln)
N_{TH}	number of through lanes (shared or exclusive) (ln)
N_{tot}	total number of circulating pedestrians arriving each cycle (p)
N_{tr}	number of transit routes along the subject segment
N_{ts}	average number of transit stops along the subject route (stops)
N_{turn}	number of lanes in the turn bay (ln)
N_{tv}	number of turning vehicles during the walk and pedestrian clear intervals (veh)
N_{WL}	number of lanes from which a weaving maneuver may be completed with one lane change or no lane changes
n_x	number of calls necessary to extend the green to max out
$O_{a,i}$	adjusted volume for origin *i* (veh/h)
OCC_{bicg}	bicycle occupancy
OCC_{pedu}	pedestrian occupancy after the opposing queue clears
OCC_r	relevant conflict-zone occupancy
O_i	volume for origin *i* (veh/h)
P	proportion of vehicles arriving during the green indication
p'	adjustment to the major-street left, minor-street through impedance factor
$p_{ap,lt}$	proportion of $N_{ap,o}$ that can be accessed by a left turn from the subject direction of travel
p_b	proportion of time blocked (decimal)
$p_{b,x}$	proportion of time that the subject movement *x* is blocked by the major-street platoon
P_{BCDEF}	probability that an individual will respond with a score of B, C, D, E, or F
p_{be}	proportion of stops on segment with benches (decimal)
P_{bo}	probability of two blocked lanes in the opposing direction
P_{bs}	probability of two blocked lanes in the subject direction
p_{building}	proportion of sidewalk length adjacent to a building face (decimal)
P_c	pavement condition rating
PC	pedestrian clear setting (s)
P_{CDEF}	probability that an individual will respond with a score of C, D, E, or F
p_{curb}	proportion of segment with curb on the right-hand side (decimal)
P_{DEF}	probability that an individual will respond with a score of D, E, or F
P_{do}	probability of delayed passing in opposing direction
P_{ds}	probability of delayed passing in subject direction
P_{EF}	probability that an individual will respond with a score of E or F
P_F	probability that an individual will respond with a score of F
P_{FD}	proportion of diverging traffic remaining in Lanes 1 and 2 immediately upstream of the deceleration lane
p_{fence}	proportion of sidewalk length adjacent to a fence or low wall (decimal)
P_{FM}	proportion of freeway vehicles remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area
PHF	peak hour factor

P_{HV}	percentage of heavy vehicles in the corresponding movement group (%)
P_{HVa}	adjusted percentage of heavy vehicles in the midsegment demand flow rate (%)
p_i	path mode split for user group i
$p_{i,j}$	seed proportion of volume from origin i to destination j (decimal)
p_j	distance required to pass mode i (mi)
P_L	proportion of left-turning vehicles in the shared lane
P_{lc}	probability of a lane change among the approach through lanes
P_{lt}	proportion of left-turning vehicles on the subject approach (decimal)
P_{LT}	proportion of left-turning vehicles in the lane
$P_{LTL,seg}$	proportion of intersections with left-turn lanes (or bay) on segment (decimal)
P_{lto}	proportion of left-turning vehicles in the opposing traffic stream
P_{mds}	probability of delayed passing for mode m
P_{no}	probability of blocked lane in opposing direction
P_{ns}	probability of blocked lane in subject direction
p_{ot}	proportion of transit vehicles arriving on time (decimal)
p_{ov}	probability of left-turn bay overflow (decimal)
p_{pk}	proportion of on-street parking occupied (decimal)
P_R	proportion of recreational vehicles in the traffic stream (Chapter 10); proportion of right-turning vehicles in the shared lane (Chapter 30)
p_{rm}	proportion of link length with restrictive median (decimal)
P_{rt}	proportion of right-turning vehicles on the subject approach (decimal)
P_{RT}	proportion of right-turning vehicles in the lane
p_{sh}	proportion of stops on segment with shelters (decimal)
P_T	proportion of trucks and buses in the traffic stream
PT	passage time setting (s)
P_{TC}	proportion of trucks operating at crawl speed (decimal)
P_{Tds}	total probability of delayed passing
PT_{lt}	passage time setting for phase serving left-turning vehicles (s)
PT_{rt}	passage time setting for phase serving right-turning vehicles (s)
$PTSF_d$	percent time-spent-following in the analysis direction (decimal)
$PTSF_F$	percent time-spent-following for the facility (decimal)
$PTSF_i$	percent time-spent-following for segment i (decimal)
$PTSF_{pl}$	percent time-spent-following for segment as affected by the presence of a passing lane (decimal)
PT_{th}	passage time setting for phase serving through vehicles (s)
P_{turn}	proportion of turning vehicles in the shared lane = P_L or P_R (decimal)
P_{window}	proportion of sidewalk length adjacent to a window display (decimal)
p_x	probability of phase termination by extension to the maximum green limit
q	arrival flow rate = v/3,600 (veh/s)
Q	back-of-queue size (veh/ln)
q^*	arrival flow rate for the phase (veh/s)
$q'_{a\|u,j}$	arrival flow rate in time step j at a downstream intersection from upstream source u (veh/step)
$q'_{u,i}$	departure flow rate in time step i at upstream source u (veh/step)

Q_1 first-term back-of-queue size (veh/ln)

Q_2 second-term back-of-queue size (veh/ln)

$Q_{2,sl}$ second-term back-of-queue size for shared left-turn and through lane group (veh/ln)

$Q_{2,sr}$ second-term back-of-queue size for shared right-turn and through lane group (veh/ln)

$Q_{2,t}$ second-term back-of-queue size for exclusive through lane group (veh/ln)

Q_{2+3} back-of-queue size (veh/ln)

Q_3 third-term back-of-queue size (veh/ln)

$Q_{3,sl}$ third-term back-of-queue size for shared left-turn and through lane group (veh/ln)

$Q_{3,sr}$ third-term back-of-queue size for shared right-turn and through lane group (veh/ln)

$Q_{3,t}$ third-term back-of-queue size for exclusive through lane group (veh/ln)

Q_{95} 95th percentile queue (veh)

Q_A estimated average per lane queue length for the through movement in the downstream (internal) link at the beginning of upstream arterial Phase A (ft)

Q_b initial queue at the start of the analysis period (veh)

$Q_{b,comb}$ initial queue for the combined movement (veh)

q_d arrival flow rate for downstream lane group (veh/s)

Q_e queue at the end of the analysis period (veh)

Q_{eo} queue at the end of the analysis period when $v \geq c_A$ and $Q_b = 0.0$ (veh)

Q_f queue size at the end of g_f (veh)

q_g arrival flow rate during the effective green time (veh/s)

q_i hourly directional path flow rate for user group i (modal users/h)

Q_i queue size at the end of interval i (veh)

$Q_{INITIAL}$ length of the queue stored at the internal approach at the beginning of the interval during which this approach has demand starvation potential

q_n outside lane flow rate = v_n/3,600 (veh/s)

Q_{ob} bicycle demand in the opposing direction (bicycles/h)

Q_p queue size at the end of permitted service time (veh)

Q_p' queue size at the end of permitted service time, adjusted for sneakers (veh)

Q_q queue size at the start of g_u (veh)

q_r arrival flow rate during the effective red time (veh/s)

Q_r queue size at the end of effective red time (= $q_r\, r$) (veh)

Q_R estimated average per lane queue length for the through movement in the downstream (internal) link at the beginning of upstream ramp Phase R (ft)

Q_{sb} bicycle demand in the same direction (bicycles/h)

Q_{sep} average queue length for the movement considered as a separate lane (veh)

Q_T total hourly directional path demand (modal users/h)

Q_{tdo} total time spent by pedestrians waiting to cross the major street during one cycle (p-s)

r effective red time (= $C - g$) (s)

R manual adjustment for on-ramps (veh/h, Chapter 10); radius of corner curb (Chapter 18); critical flow ratio for the exit-ramp movements (Chapter 22)

R, b, c intermediate calculation variables

r_a acceleration rate = 3.5 (ft/s^2)

r_{at} transit vehicle acceleration rate (ft/s^2)

R_c red clearance interval (s)

$R_{c,mi}$	red clearance interval of the phase serving the minor-street through movement (s)
r_d	deceleration rate
r_{dt}	transit vehicle deceleration rate (ft/s²)
R_p	platoon ratio
R_Q	queue storage ratio
r_{qg}	queue growth rate (veh/h)
RS	reference sum flow rate (1,530 × PHF × f_a) (veh/h)
RW	reciprocal of path width = 1/path width (ft)
s	saturation flow rate (veh/h, Chapter 4); mean service rate (veh/h, Chapter 4); standard deviation of the set of runs for a particular performance measure (Chapter 7); adjusted saturation flow rate (veh/h/ln, Chapter 17)
S	average travel speed (mi/h)
s_0	base saturation flow rate (veh/hg/ln)
S_0	speed constant (mi/h)
S_{0i}	free-flow speed of segment i (mi/h)
s_1	saturation flow rate for the inside lane (veh/h/ln)
$S_{85,mj}$	85th percentile speed at a midsegment location on the major street (mi/h)
S_a	average speed on the intersection approach (mi/h)
S_b	bicycle running speed (mi/h)
SF	service flow rates
S_f	free-flow speed (mi/h)
S_F	free-flow speed of the ramp at the junction point (mi/h)
SF_i	service flow rate for LOS i (veh/h)
SFI_i	service flow rate under ideal conditions (pc/h)
S_{FM}	mean speed of sample (v > 200 veh/h) (mi/h)
S_{fo}	base free-flow speed (mi/h)
S_{FR}	free-flow speed of the ramp (mi/h)
S_i	average vehicle speed on segment i (mi/h)
s_i	saturation flow rate for phase movement i (veh/hg/ln)
s_l	saturation flow rate in exclusive left-turn lane group with permitted operation (veh/h/ln)
s_{lc}	maximum flow rate in which a lane change can occur = 3,600/t_{lc} (veh/h/ln)
s_{lr}	saturation flow rate in shared left- and right-turn lane group (veh/h/ln)
s_{lt}	saturation flow of an exclusive left-turn lane with protected operation (veh/h/ln)
S_{MAX}	maximum average speed of weaving vehicles expected in a weaving segment (mi/h)
S_{MIN}	minimum average speed of weaving vehicles expected in a weaving segment (mi/h)
S_{NW}	average speed of nonweaving vehicles within the weaving segment (mi/h)
s_o	base saturation flow rate (pc/h/ln)
S_O	average speed of vehicles in outer lanes of the freeway, adjacent to the 1,500-ft ramp influence area (mi/h)
$s_{o,local}$	local base saturation flow rate (pc/h/ln)
s_p	saturation flow rate of a permitted left-turn movement (veh/h/ln)
S_p	posted speed limit (mi/h, Chapter 15); pedestrian walking speed = 3.5 ft/s (Chapter 30)

S_{ped} pedestrian speed (ft/min)

S_{pf} free-flow pedestrian walking speed (ft/s)

S_{pl} posted speed limit (mi/h)

$s_{prevaling,i}$ prevailing saturation flow rate for lane group i (veh/h/ln)

$s_{q|r}$ shared lane discharge flow rate for upstream right-turn traffic movement (veh/h/ln)

s_r saturation flow rate in exclusive right-turn lane group with permitted operation (veh/h/ln)

S_R average speed in the ramp influence area (mi/h)

S_{Ra} adjusted motorized vehicle running speed (mi/h)

S_{Rt} transit vehicle running speed (mi/h)

S_s threshold speed defining a stopped vehicle (= 5.0 mi/h)

s_{sl} saturation flow rate in shared left-turn and through lane group with permitted operation (veh/h/ln)

$s_{sl,min}$ minimum saturation flow rate in shared left-turn and through lane group with permitted operation (veh/h/ln)

s_{sl2} saturation flow rate in shared left-turn and through lane group during Period 2 (veh/h/ln)

S_{spot} average spot speed (mi/h)

s_{sr} saturation flow rate in shared right-turn and through lane group with permitted operation (veh/h/ln)

s_t saturation flow rate in exclusive through lane group (veh/h/ln)

S_t effective speed factor

$S_{T,seg}$ travel speed of through vehicles for the segment (mi/h)

$S_{Tb,seg}$ travel speed of through bicycles along the segment (mi/h)

s_{th} saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, and area type) (veh/h/ln)

$S_{Tp,seg}$ travel speed of through pedestrians for the segment (ft/s)

$S_{Tt,seg}$ travel speed of transit vehicles along the segment (mi/h)

SV_i service volume for LOS i (veh/h)

S_W average speed of weaving vehicles within the weaving segment (mi/h)

s_{w-r} transit wait–ride score

t duration of unmet demand in the analysis period (h, Chapter 18); path segment travel time for average bicycle (min, Chapter 23)

T analysis time period

t' platoon arrival time (steps)

t'_p blocked period duration (steps)

t'_R segment running time (steps)

T_0 time at which a vehicle would have arrived at the stop line if it had been traveling at the reference speed

T_1 time at which a vehicle would have arrived at the stop line if it had been traveling at the “running” speed

T_2 time at which a vehicle is discharged at the stop line

$t_{3,LT}$ adjustment factor for intersection geometry

t_a average duration of unmet demand in the analysis period (h)

t_A adjusted duration of unmet demand in the analysis period (h)

T_{at} amenity time rate (min/mi)

T_{btt} base travel time rate = 6.0 for the central business district of a metropolitan area with 5 million persons or more, otherwise 4.0 (min/mi)

T_c time until spillback (h)

t_c queue clearing time (Chapter 18); critical headway for a single pedestrian (s, Chapter 19)

$t_{c,base}$ base critical headway (s)

$t_{c,G}$ adjustment factor for grade (s)

$t_{c,HV}$ adjustment factor for heavy vehicles (s)

$t_{c,x}$ critical headway (i.e., the minimum time that allows intersection entry for one minor-stream vehicle) for minor movement x (s)

t_{cg} critical gap (s)

t_{cl} clearance time of the right-turn vehicle (s)

t_d dwell time (s)

$t_{d,i}$ duration of time interval i during which the arrival flow rate and saturation flow rate are constant (s)

t_{ex} excess wait time due to late arrivals (s)

T_{ex} excess wait time rate due to late arrivals (min/mi)

t_f follow-up headway (s, Chapter 21); service time for fully stopped vehicles (s, Chapter 30)

$t_{f,base}$ base follow-up headway (s)

$t_{f,HV}$ adjustment factor for heavy vehicles (0.9 for major streets with one lane in each direction, 1.0 for major streets with two or three lanes in each direction)

$t_{f,x}$ follow-up headway (i.e., the time between the departure of one vehicle from the minor street and the departure of the next vehicle under a continuous queue condition) for minor movement x (s)

t_{fh} follow-up headway (s)

t_i lost time for ith vehicle in queue (s, Chapter 4); duration of unmet demand for lane group i in the analysis period (h, Chapter 18)

t_l transit vehicle running time loss (min/mi)

t_L lost time per phase (s)

t_L' adjusted lost time (i.e., the time during which the signalized intersection is not used effectively by any movement) (s)

t_L'' adjusted lost time for the internal approaches (s)

t_{late} threshold late time = 5.0 typical (min)

t_{lc} critical merge headway = 3.7 (s)

T_{LC} total lateral clearance (ft)

T_{occ} crosswalk occupancy time (p-s)

T_p analysis time period

t_{pr} pedestrian perception of signal indication and curb departure time (= 7.0 s)

t_{ps} pedestrian service time (s)

T_{ptt} perceived travel time rate (min/mi)

t_Q time duration of queue (s)

t_R segment running time (s)

t_{Rb} segment running time of through bicycles (s)

T_{RD} total ramp density (ramps/mi)

t_{Rt} segment running time of transit vehicle (s)

t_s	pedestrian start-up time and end clearance time (s, Chapter 19); service time or average time spent by a vehicle in first position waiting to depart (s, Chapter 20)
TS_c	time-space available for circulating pedestrians (ft²-s)
TS_{corner}	available corner time-space (ft²-s)
TS_{cw}	available crosswalk time-space (ft²-s)
TS_{cw}^*	effective available crosswalk time-space (ft²-s)
TS_{tv}	time-space occupied by turning vehicles (ft²-s)
$t_{t,i}$	duration of trapezoid or triangle in interval i (s)
TT_i	total travel time of all vehicles in segment i (veh-h)
TT_{i15}	total travel time consumed by all vehicles traversing directional segment i during the 15-min analysis period (veh-h)
U	speed of average bicycle (mi/h)
u_m	minimum speed of the first through vehicle given that it is delayed (ft/s)
u_{rt}	right-turn speed (ft/s)
v	actual flow rates
V	demand volume under prevailing conditions (veh/h)
v_1	flow rate for the inside lane (veh/h/ln)
v_{12}	demand flow rate in Lanes 1 and 2 of the freeway immediately upstream of the ramp influence area (pc/h)
v_{15}	pedestrian flow rate during the peak 15 min (p/h)
V_{15}	volume during the peak 15 min of the analysis hour (veh/15 min)
v_2	flow rate in the adjacent through lane (veh/h/ln)
v_3	flow rate in Lane 3 of the freeway (pc/h/ln)
$v_{a,1}$	adjusted arrival volume in the shared lane (veh/h)
$v_{a,b}$	flow rate of pedestrians traveling through the corner from Sidewalk A to Sidewalk B, or vice versa (p/h)
$v_{A,x}$	volume or flow rate on approach x (veh/h)
v_a/N_A	arterial flow feeding the subject queue, divided by the respective number of lanes
v_{app}	average demand flow rate per through lane (upstream of any turn bays on the approach) (veh/h/ln)
$v_{Arterial}$	upstream arterial through flow (v/h)
v_{bic}	bicycle flow rate (bicycles/h)
v_{bicg}	bicycle flow rate during the green indication (bicycles/h)
v_c	conflicting flow rate
$v_{c,min}$	minimum platooned flow rate (veh/h), assumed to be 1,000N, where N is the number of through lanes per direction on the major street
$v_{c,pce}$	conflicting flow rate (pc/h)
$v_{c,u,x}$	conflicting flow for movement x during the unblocked period (veh/h)
$v_{c,x}$	conflicting flow rate for movement x (veh/h)
v_{ci}	flow rate of pedestrians arriving at the corner after crossing the minor street (p/h)
v_{co}	flow rate of pedestrians arriving at the corner to cross the minor street (p/h)
v_D	flow rate on the adjacent downstream ramp (pc/h)
$v_{d,ATS}$	demand flow rate for ATS determination in the analysis direction (pc/h)
$v_{d,PTSF}$	demand flow rate in the analysis direction for estimation of PTSF (pc/h)

v_{di} flow rate of pedestrians arriving at the corner after crossing the major street (p/h)

v_{do} flow rate of pedestrians arriving at the corner to cross the major street (p/h)

$V_{dOFF15ij}$ adjusted 15-min exit demand for time period *i* and exiting location *j* (veh)

v_e entry flow rate

v_{ex} exit flow rate

$v_{ex,pce}$ conflicting exiting flow rate (pc/h)

v_F flow rate on freeway immediately upstream of the ramp influence area under study (pc/h)

v_{FF} freeway-to-freeway demand flow rate in the weaving segment (pc/h)

v_{FO} flow rate on the freeway immediately downstream of the merge or diverge area (pc/h)

v_{FR} freeway-to-ramp demand flow rate in the weaving segment (pc/h)

v_g demand flow rate for movement group (veh/h)

v_{g1} demand flow rate in the single exclusive lane with the highest flow rate of all exclusive lanes in movement group (veh/h/ln)

v_h pedestrian demand during the analysis hour (p/h)

v_i demand flow rate for movement *i* (pc/h, Chapter 19); phase movement volume for phase *i* (veh/h, Chapter 22)

V_i demand volume for movement *i* (veh/h, Chapter 19); speed of path user of mode *i* (mi/h, Chapter 23)

v_i' demand flow rate (veh/cycle/ln)

$v_{i,1}$ major-street through vehicles in shared lane (veh/h)

$v_{i,2}$ major-street turning vehicles in shared lane (veh/h)

$v_{i,ATS}$ demand flow rate *i* for ATS estimation (pc/h)

$v_{i,j}$ volume entering from origin *i* and exiting at destination *j* (veh/h)

$v_{i,pce}$ demand flow rate for movement *i* (pc/h)

$v_{i,PTSF}$ demand flow rate *i* for determination of PTSF (pc/h)

v_j flow rate of movement *j*

v_l demand flow rate in exclusive left-turn lane group (veh/h/ln)

v_L major left-turn/U-turn flow rate (veh/h)

v_{L+TH} through and left-turn volume (veh/h)

v_{lr} demand flow rate in shared left- and right-turn lane group (veh/h)

v_{lt} left-turn demand flow rate (veh/h)

$v_{lt,perm}$ permitted left-turn demand flow rate (veh/h)

v_m midsegment demand flow rate (direction nearest to the subject sidewalk) (veh/h)

v_{ma} adjusted midsegment demand flow rate (veh/h)

v_{mg} merge flow rate (veh/h/ln)

VMT_i vehicle miles traveled for segment *i* (veh-mi)

VMT_{i15} total vehicle miles traveled by all vehicles in directional segment *i* during the 15-min analysis period (veh-mi)

v_n flow rate for the outside lane (veh/h/ln)

v_{NW} nonweaving demand flow rate in the weaving segment (pc/h)

v_o opposing demand flow rate (veh/h)

$v_{o,ATS}$ demand flow rate for ATS determination in the opposing direction (pc/h)

$v_{o,PTSF}$ demand flow rate in the opposing direction for estimation of PTSF (pc/h)

v_{OA} average per lane demand flow in outer lanes adjacent to the ramp influence area (not including flow in Lanes 1 and 2) (pc/h/ln)

$V_{OFF15ij}$ 15-min exit count for time period *i* and exiting location *j* (veh)

v_{OL} directional demand flow rate in the outside lane (veh/h)

V_{ON15ij} 15-min entering count for time period *i* and entering location *j* (veh)

v_p demand flow rate under equivalent base conditions (pc/h/ln, Chapter 11); pedestrian flow per unit width (p/ft/min, Chapter 17)

v_{ped} pedestrian flow rate (p/min/ft)

$v_{ped,i}$ pedestrian flow rate in the subject crossing for travel direction *i* (p/h)

v_{pedg} pedestrian flow rate during the pedestrian service time (p/h)

v_R flow rate on the on-ramp or off-ramp (pc/h, Chapter 13); right-turn volume (veh/h, Chapter 19)

VR volume ratio

v_R/N_R ramp flow that feeds the subject queue, divided by the respective number of lanes

v_{R12} sum of the flow rates in Lanes 1 and 2 and the ramp flow rate (on-ramps only) (pc/h)

$v_{Ramp\text{-}L}$ upstream ramp left-turning flow (v/h)

v_{RF} ramp-to-freeway demand flow rate in the weaving segment (pc/h)

v_{RR} ramp-to-ramp demand flow rate in the weaving segment (pc/h)

v_{rt} right-turn demand flow rate (veh/h)

v_{rtor} right-turn-on-red flow rate (veh/h)

v_s transit frequency for the segment (veh/h)

v_{sep} flow rate for the movement (veh/h)

v_{sl} demand flow rate in shared left-turn and through lane group (veh/h)

$v_{sl,lt}$ left-turn flow rate in shared lane group (veh/h/ln)

v_{sr} demand flow rate in shared right-turn and through lane group (veh/h)

$v_{sr,rt}$ right-turn flow rate in shared lane group (veh/h/ln)

v_t demand flow rate in exclusive through lane group (veh/h/ln, Chapter 17); transit vehicle flow rate in pattern = $\sum v_i$ (veh/h, Chapter 17)

v_{th} through demand flow rate (veh/h)

V_{tot} total number of vehicles arriving during the survey period (veh)

v_U flow rate on the adjacent upstream ramp (pc/h)

v_W weaving demand flow rate in the weaving segment (pc/h)

v_x flow rate for movement *x* (veh/h)

v_y flow rate of the *y* movement in the subject shared lane (veh/h)

w lane width of the lane that the minor movement is negotiating into (ft)

W weaving intensity factor (Chapter 12); effective width of crosswalk (ft, Chapter 30)

W_1 effective width of combined bicycle lane and shoulder (ft)

W_a effective width of Sidewalk A (ft)

W_A available sidewalk width (ft)

W_{aA} adjusted available sidewalk width (ft)

$Walk$ pedestrian walk setting (s)

W_b effective width of Sidewalk B (ft)

W_{bl} width of the bicycle lane (ft)

W_{buf} buffer width between roadway and available sidewalk (ft)

W_{cd} curb-to-curb width of the cross street (ft)

W_d effective width of Crosswalk D (ft)

W_e average effective width of the outside through lane (ft)

W_E effective crosswalk width (ft)

W_i width of signalized intersection as measured along the segment centerline (ft)

W_O sum of fixed-object effective widths and linear-feature shy distances at a given point along the walkway (ft)

$W_{O,i}$ adjusted fixed-object effective width on inside of sidewalk (ft)

$W_{O,o}$ adjusted fixed-object effective width on outside of sidewalk (ft)

W_{ol} width of the outside through lane (ft)

W_{OL} outside lane width (ft)

W_{os} width of paved outside shoulder (ft)

w_q queue change rate (veh/s)

W_s paved shoulder width (ft)

$W_{s,i}$ shy distance on inside of sidewalk (ft)

$W_{s,o}$ shy distance on outside of sidewalk (ft)

W_t total width of the outside through lane, bicycle lane, and paved shoulder (ft)

W_T total walkway width at a given point along the walkway (ft)

W_v effective width as a function of traffic volume (ft)

x degree of utilization

X volume-to-capacity ratio

X_1 volume-to-capacity ratio in the shared lane

X_A average volume-to-capacity ratio

X_c critical intersection volume-to-capacity ratio

X_i volume-to-capacity ratio for lane group *i*

X_u weighted volume-to-capacity ratio for all upstream movements contributing to the volume in the subject movement group

Y yellow-plus-all-red change-and-clearance interval (s)

Y_c sum of the critical flow ratios

$y_{c,i}$ critical flow ratio for phase *i*

Y_{mi} change interval of the phase serving the minor-street through movement (s)

YP_2 yield point for Phase 2 (s)

y_t effective flow ratio for the concurrent phase when dictated by travel time

Δ headway of bunched vehicle stream (s/veh)

Δ^* equivalent headway of bunched vehicle stream served by the phase (s/veh)

Δ_i headway of bunched vehicle stream in lane group *i* (s/veh)

λ flow rate parameter (veh/s)

λ^* flow rate parameter for the phase (veh/s)

$\lambda_{c,i}$ flow rate parameter for lane group *i* served in the concurrent phase that also ends at the barrier (veh/s)

λ_i flow rate parameter for lane group *i*

λ_l flow rate parameter for the exclusive left-turn lane group (veh/s)

λ_r flow rate parameter for the exclusive right-turn lane group (veh/s)

λ_{sl} flow rate parameter for shared left-turn and through lane group (veh/s)

λ_{sr}	flow rate parameter for shared right-turn and through lane group (veh/s)
λ_t	flow rate parameter for exclusive through lane group (veh/s)
σ_{spot}	standard deviation of spot speeds (mi/h)
ΣV_{iq}	sum of vehicle-in-queue counts (veh)
φ^*	combined proportion of free (unbunched) vehicles for the phase
φ_i	proportion of free (unbunched) vehicles in lane group i

T

VOLUME 2
UNINTERRUPTED FLOW

OVERVIEW

Volume 2 of the *Highway Capacity Manual* (HCM) contains six chapters that present analysis methods for uninterrupted-flow roadways—that is, roadways that have no fixed causes of delay or interruption external to the traffic stream. This volume addresses three types of uninterrupted-flow roadways:

- *Freeways,* defined as separated highways with full control of access and two or more lanes in each direction dedicated to the exclusive use of traffic;
- *Multilane highways,* defined as highways that do not have full control of access and that have two or more lanes in each direction, with traffic signals or roundabouts spaced at least 2 mi apart on average; and
- *Two-lane highways,* defined as roadways with one lane for traffic in each direction (except for occasional passing lanes or truck climbing lanes), with traffic signals, roundabouts, or STOP-controlled intersections spaced at least 2 mi apart on average.

The HCM treats roadways that have traffic signals, roundabouts, or STOP-controlled intersections spaced less than 2 mi apart on average as urban streets. Urban streets are discussed in Volume 3, Interrupted Flow.

VOLUME ORGANIZATION

Freeways

Traffic enters and exits a freeway via ramps. **Chapter 13, Freeway Merge and Diverge Segments,** focuses on locations where two or more traffic streams combine to form a single traffic stream (a *merge*) or where a single traffic stream divides to form two or more separate traffic streams (a *diverge*). These locations are most commonly ramp–freeway junctions but include points where mainline roadways join or separate. Chapter 13 can also be applied in an approximate way to ramp–highway junctions on multilane highways and collector–distributor roads. Ramp–street junctions are analyzed with the methods in the intersection and interchange chapters in Volume 3.

Sometimes freeway merges are closely followed by freeway diverges, or a one-lane off-ramp closely follows a one-lane on-ramp and the two are connected by a continuous auxiliary lane. In these cases, the traffic streams to and from the ramps must cross each other over a significant length of freeway without the aid of traffic control devices (except for guide signs). The term "closely" implies that the distance between the merge and diverge segments is not sufficient for them to operate independently, thus creating a *weave.* **Chapter 12, Freeway Weaving Segments,** provides procedures for analyzing weaving operations on freeways.

It can be applied in an approximate way to weaves on multilane highways and collector–distributor roads, but not to weaves on arterial streets.

The remaining portions of the freeway mainline that are not merge, diverge, or weaving segments (except for toll plazas, drawbridges, or similar points where freeway traffic may be temporarily required to stop) are covered in **Chapter 11, Basic Freeway Segments.** This chapter also provides information on the base conditions and passenger car equivalents for heavy vehicles that are common to all of the freeway chapters.

Chapter 10, Freeway Facilities, provides a methodology for analyzing extended lengths of freeway composed of continuously connected basic freeway, weaving, merge, and diverge segments. Such extended lengths are referred to as a *freeway facility*. In this terminology, the term *facility* does not refer to an entire freeway from beginning to end; instead, it refers to a specific set of connected segments that have been identified for analysis. In addition, the term does not refer to a freeway system consisting of several interconnected freeways.

The methodologies of Chapters 11, 12, and 13 all focus on a single time period of interest, generally the peak 15 min within a peak hour. However, Chapter 10's methodology allows for the analysis of multiple and continuous 15-min periods and is capable of identifying breakdowns and the impact of such breakdowns over space and time.

Multilane Highways

Chapter 14, Multilane Highways, presents analysis methods for the portions of multilane highways away from the influence of signalized intersections (or other forms of intersection traffic control that interrupt the flow of traffic on the highway). Many multilane highways will have periodic signalized intersections, even if the average signal spacing is well over 2 mi. In such cases, the multilane highway segments that are more than 2 mi away from any signalized intersections are analyzed with the Chapter 14 methodology. Isolated signalized intersections should be analyzed with the methodology of Chapter 18, Signalized Intersections.

Bicycles are typically permitted on multilane highways, and multilane highways often serve as primary routes for both commuter cyclists (on suburban highways) and recreational cyclists (on rural highways). Chapter 14 presents a method for estimating the bicycle level of service (LOS) on multilane highways.

Two-Lane Highways

Chapter 15, Two-Lane Highways, presents analysis methods for the portions of two-lane highways that are away from the influence of intersection traffic control that interrupts the flow of traffic. In general, any segment that is 2.0 to 3.0 mi from the nearest signalized intersection, roundabout, or intersection where the highway is STOP-controlled would fit into this category. Where these interruptions to traffic are less than 2.0 mi apart, the facility should be classified as an urban street and analyzed with the methodologies of Chapter 16, Urban Street Facilities, and Chapter 17, Urban Street Segments, which are located in Volume 3.

Chapter 15 can be used to analyze three classes of two-lane highways:

- *Class I* highways are ones where motorists expect to travel at relatively high speeds, such as major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks;
- *Class II* highways are ones where motorists do not necessarily expect to travel at high speeds, such as highways serving as access routes to Class I facilities, serving as scenic or recreational routes, or passing through rugged terrain; and
- *Class III* highways are ones serving moderately developed areas, such as portions of a Class I or Class II highway passing through small towns or developed recreational areas or longer segments passing through more spread-out recreational areas, with increased roadside densities.

Two-lane highways often serve as routes for recreational cyclists. Chapter 15 presents a method for estimating the bicycle LOS on these highways.

RELATED CHAPTERS

Volume 1

The chapters in Volume 2 assume that the reader is already familiar with the concepts presented in the Volume 1 chapters, in particular the following:

- *Chapter 2, Applications*—types of HCM analysis, types of roadway system elements, and traffic flow characteristics;
- *Chapter 3, Modal Characteristics*—variations in demand, peak and analysis hours, *K*- and *D*-factors, facility types by mode, and interactions between modes;
- *Chapter 4, Traffic Flow and Capacity Concepts*—traffic flow parameters and factors that influence capacity; and
- *Chapter 5, Quality and Level-of-Service Concepts*—performance measures, service measures, and LOS.

Volume 3

The intersection and interchange chapters (Chapters 18–22) are used to determine the operations of freeway ramp–street junctions and the operations of isolated traffic signals, roundabouts, and STOP-controlled intersections along multilane and two-lane highways. In the context of Volume 2, it is particularly important to examine the length of the queue extending back from a freeway off-ramp–street junction, since long queues may affect freeway operations, a situation that is not accounted for in the HCM techniques.

Volume 4

Five chapters in Volume 4 (accessible at www.HCM2010.org) provide additional information that supplements the material presented in Volume 2. These chapters are as follows:

- *Chapter 25, Freeway Facilities: Supplemental*—details of the computations used in the Chapter 10 methodology, and computational engine flowcharts and linkage lists;
- *Chapter 26, Freeway and Highway Segments: Supplemental*—examples of applying alternative tools to situations that are not addressed by the Chapter 11 method for basic freeway segments, and state-specific default values for heavy vehicle percentage that apply to all Volume 2 chapters;
- *Chapter 27, Freeway Weaving: Supplemental*—examples of applying alternative tools to situations not addressed by the Chapter 12 method;
- *Chapter 28, Freeway Merges and Diverges: Supplemental*—examples of applying alternative tools to situations not addressed by the Chapter 13 method; and
- *Chapter 35, Active Traffic Management*—descriptions of active traffic management strategies; a discussion of the mechanisms by which they affect demand, capacity, and performance; and general guidance on possible evaluation methods for active traffic management techniques.

Access Volume 4 at www.HCM2010.org

The *HCM Applications Guide* in Volume 4 provides three case studies on the analysis of uninterrupted-flow facilities:

- *Case Study No. 3* illustrates the process of applying HCM techniques to the analysis of a two-lane highway;
- *Case Study No. 4* illustrates the process of applying HCM techniques to the analysis of a freeway; and
- *Case Study No. 6* illustrates the application of alternative tools to a freeway facility in a situation where HCM techniques are unsuitable.

Case Studies No. 3 and No. 4 focus on the process of applying the HCM rather than on the details of performing calculations (which are addressed by the example problems in the Volume 2 chapters). These case studies' computational results were developed by using HCM2000 methodologies and therefore may not match the results obtained from applying the HCM 2010. However, the process of application is the focus, not the specific computational results.

The Technical Reference Library in Volume 4 contains copies of (or links to) many of the documents referenced in Volume 2 and its supplemental chapters. Because the Chapter 10 methodology is too complex to be implemented by manual pencil-and-paper techniques, the FREEVAL-2010 spreadsheet has been developed to implement the methodology's calculations. The Technical Reference Library contains a copy of the spreadsheet along with a user's guide.

LEVELS OF ANALYSIS AND ANALYSIS TOOLS

As discussed in Chapter 2, Applications, HCM methodologies can be applied to the operations, design, preliminary engineering, and planning levels of analysis. These levels differ both in the amount of field data used in the analysis (as opposed to default values) and in the way the HCM is applied (iteratively, to find a design that meets a desired set of criteria, or as a single application, to evaluate performance given a particular set of inputs). Each Volume 2 chapter provides a section that discusses how to apply the chapter to these different levels of analysis, along with a section with recommended default values for planning and preliminary engineering analyses.

Three Volume 2 chapters (10, 14, and 15) provide generalized service volume tables applicable to freeway facilities, multilane highways, and two-lane highways, respectively. These tables can be used for large-scale planning efforts when the goal is to analyze a large number of facilities to determine where problems might exist or arise or where improvements might be needed. Any facilities identified as likely to experience problems or need improvement should then be subjected to a more detailed analysis that takes into account the existing or likely future characteristics of the specific facility before any detailed decisions on implementing specific improvements are made. Because the service volumes provided in these tables are highly dependent on the default values assumed as inputs, it is recommended that users wishing to apply generalized service volume tables develop their own tables by using local default values, in accordance with the processes described in Appendix A and Appendix B of Chapter 6, HCM and Alternative Analysis Tools.

Chapter 6 also describes in general terms the conditions under which the use of alternative tools to supplement HCM capacity and quality-of-service procedures should be considered. Each Volume 2 chapter contains a section discussing the potential application of alternative tools to the specific system element addressed by the chapter, and Chapters 26–28 in Volume 4 provide example problems illustrating applications of alternative tools to address HCM limitations. Each chapter lists the specific limitations of its methodology. The major limitations are summarized as follows:

- *Freeways*
    - Operations of oversaturated freeway segments (but not necessarily oversaturated freeway facilities, as discussed later)
    - Multiple overlapping breakdowns or bottlenecks
    - Conditions where off-ramp queues extend back onto the freeway or affect the behavior of exiting vehicles
    - Operation of separated high-occupancy vehicle (HOV) facilities and weaving interactions between HOV and general-purpose lanes
    - Toll plaza operations
    - Ramp-metering effects

- *Multilane highways*
  - Operations during oversaturated conditions
  - The impacts of shoulder parking activity, bus stops, or significant pedestrian activity
  - Possible queuing impacts when a multilane highway segment transitions to a two-lane highway segment
  - Differences between various types of median barriers, and the difference between the impact of a median barrier and a two-way left-turn lane
  - The range of values used to develop the bicycle LOS model (although the model has been successfully applied to rural multilane highways, users should be aware that conditions on many of those highways are outside the range of values used to develop the model)
- *Two-lane highways*
  - Operations during oversaturated conditions
  - Impact of intersection traffic control on the overall facility LOS
  - The range of values used to develop the bicycle LOS model (although the model has been successfully applied to rural two-lane highways, users should be aware that conditions on many of those highways are outside the range of values used to develop the model)

If an analysis of an individual freeway segment reveals the segment to be oversaturated, then Chapter 10, Freeway Facilities, must be used to assess operation of the segment and its impacts on upstream and downstream sections. If the Chapter 10 analysis reveals that the oversaturation would extend beyond the geographic or temporal boundaries of the analysis, then the boundaries of the Chapter 10 analysis should be expanded to contain the oversaturation. If expanding the boundaries of the analysis is not practical, then no analytical tool, including the HCM, can give a complete answer in this situation.

CHAPTER 10
FREEWAY FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

A freeway is a separated highway with full control of access and two or more lanes in each direction dedicated to the exclusive use of traffic. Freeways are composed of various uniform segments that may be analyzed to determine capacity and level of service (LOS). Three types of segments are found on freeways:

- *Freeway merge and diverge segments:* Segments in which two or more traffic streams combine to form a single traffic stream (merge) or a single traffic stream divides to form two or more separate traffic streams (diverge).
- *Freeway weaving segments:* Segments in which two or more traffic streams traveling in the same general direction cross paths along a significant length of freeway without the aid of traffic control devices (except for guide signs). Weaving segments are formed when a diverge segment closely follows a merge segment or when a one-lane off-ramp closely follows a one-lane on-ramp and the two are connected by a continuous auxiliary lane.
- *Basic freeway segments:* All segments that are not merge, diverge, or weaving segments.

Analysis methodologies are detailed for basic freeway segments in Chapter 11, for weaving segments in Chapter 12, and for merge and diverge segments in Chapter 13.

Chapter 10, Freeway Facilities, provides a methodology for analyzing extended lengths of freeway composed of continuously connected basic freeway, weaving, merge, and diverge segments. Such extended lengths are referred to as a *freeway facility.* In this terminology, the term *facility* does not refer to an entire freeway from beginning to end; instead, it refers to a specific set of connected segments that have been identified for analysis. In addition, the term does not refer to a freeway system consisting of several interconnected freeways.

The methodologies of Chapters 11, 12, and 13 focus on a single time period of interest, generally the peak 15 min within a peak hour. This chapter's methodology allows for the analysis of multiple and continuous 15-min time periods and is capable of identifying breakdowns and the impact of such breakdowns over space and time.

The methodology is integral with the FREEVAL-2010 model, which implements the complex computations involved. This chapter discusses the basic principles of the methodology and its application. Chapter 25, Freeway Facilities: Supplemental, provides a complete and detailed description of all the algorithms that define the methodology. The Technical Reference Library in Volume 4 contains a user's guide to FREEVAL-2010 and an executable spreadsheet that implements the methodology.

SEGMENTS AND INFLUENCE AREAS

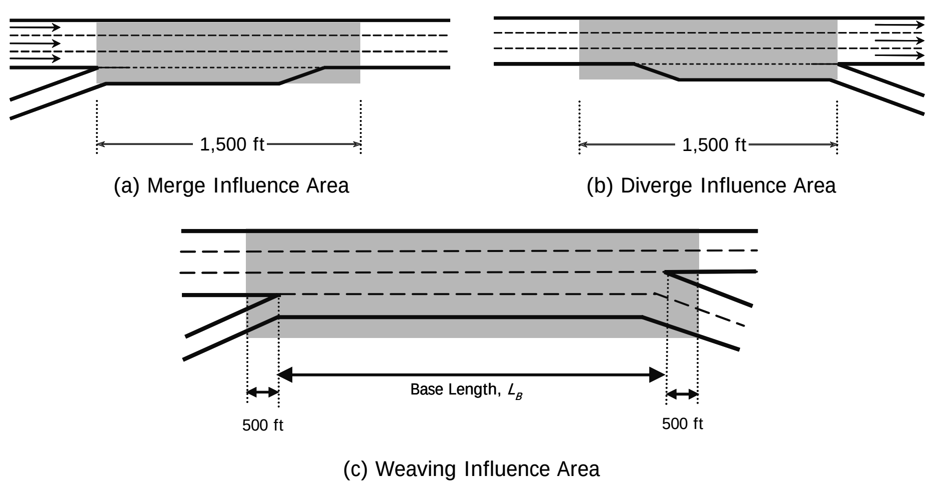
It is important that the definition of freeway segments and their influence areas be clearly understood. The influence areas of merge, diverge, and weaving segments are as follows:

- *Weaving segment:* The base length of the weaving segment plus 500 ft upstream of the entry point to the weaving segment and 500 ft downstream of the exit point from the weaving segment; entry and exit points are defined as the points where the appropriate edges of the merging and diverging lanes meet.
- *Merge segment:* From the point where the edges of the travel lanes of the merging roadways meet to a point 1,500 ft downstream of that point.
- *Diverge segment:* From the point where the edges of the travel lanes of the merging roadways meet to a point 1,500 ft upstream of that point.

Points where the "edges of travel lanes" meet are most often defined by pavement markings.

The influence areas of merge, diverge, and weaving segments are illustrated in Exhibit 10-1.

Exhibit 10-1
Influence Areas of Merge, Diverge, and Weaving Segments

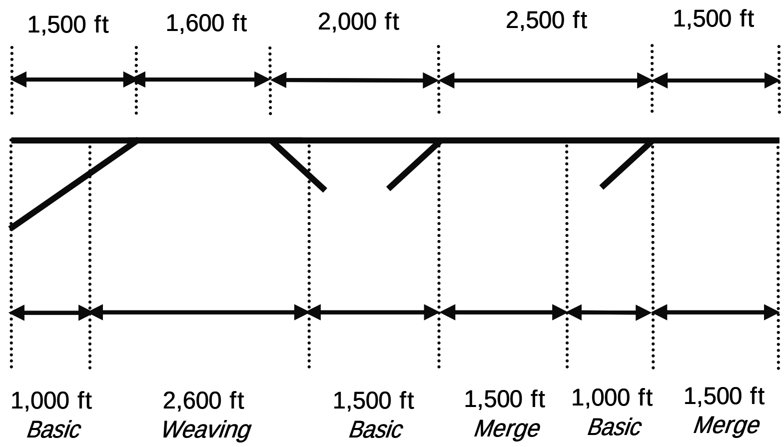


Basic freeway segments are any other segments along the freeway that are not within these defined influence areas, which is not to say that basic freeway segments are not affected by the presence of adjacent and nearby merge, diverge, and weaving segments. Particularly when a segment breaks down, its effects will propagate to both upstream and downstream segments, regardless of type. Furthermore, the general impact of the frequency of merge, diverge, and weaving segments on the general operation of all segments is taken into account by the free-flow speed of the facility.

Basic freeway segments, therefore, do exist even on urban freeways where merge and diverge points (most often ramps) are closely spaced. Exhibit 10-2 illustrates this point. It shows a 9,100-ft (1.7-mi) length of freeway with four ramp terminals, two of which form a weaving segment. Even with an average ramp spacing less than 0.5 mi, this length of freeway contains three basic freeway segments. The lengths of these segments are relatively short, but, in terms of

analysis methodologies, they must be treated as basic freeway segments. Thus, while it is true that many urban freeways will be dominated by frequent merge, diverge, and weaving segments, there will still be segments classified and analyzed as basic freeway segments.

Exhibit 10-2
Basic Freeway Segments on an Urban Freeway



FREE-FLOW SPEED

Free-flow speed is strictly defined as the theoretical speed when the density and flow rate on the study segment are both zero. Chapter 11, Basic Freeway Segments, presents speed–flow curves that indicate that the free-flow speed on freeways is expected to prevail at flow rates between 0 and 1,000 passenger cars per hour per lane (pc/h/ln). In this broad range of flows, speed is insensitive to flow rates. This characteristic simplifies and allows for measurement of free-flow speeds in the field.

Chapter 11 also presents a methodology for estimating the free-flow speed of a basic freeway segment if it cannot be directly measured. The free-flow speed of a basic freeway segment is sensitive to three variables:

- Lane widths,
- Lateral clearances, and
- Total ramp density.

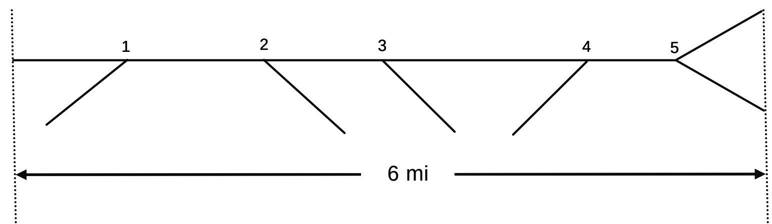
The most critical of these variables is total ramp density. *Total ramp density* is defined as the average number of on-ramp, off-ramp, major merge, and major diverge junctions per mile. It applies to a 6-mi segment of freeway facility, 3 mi upstream and 3 mi downstream of the midpoint of the study segment.

While the methodology for determining free-flow speed is provided in Chapter 11, Basic Freeway Segments, it is also applied in Chapter 12, Freeway Weaving Segments, and Chapter 13, Freeway Merge and Diverge Segments. Thus, free-flow speed affects the operation of all basic, weaving, merge, and diverge segments on a freeway facility.

The free-flow speed is an important characteristic, as the capacity *c*, service flow rates *SF*, service volumes *SV*, and daily service volumes *DSV* all depend on it.

Exhibit 10-3 illustrates the determination of total ramp density on a 6-mi length of freeway facility.

Exhibit 10-3
Ramp Density Determination



As illustrated in Exhibit 10-3, there are four ramp terminals and one major diverge point in the 6-mi segment illustrated. The total ramp density is, therefore, 5/6 = 0.83 ramp/mi.

CAPACITY OF FREEWAY FACILITIES

Capacity traditionally has been defined for segments of uniform roadway, traffic, and control conditions. When facilities consisting of a series of connected segments are considered, the concept of capacity is more complicated.

The methodologies of Chapters 11, 12, and 13 allow the capacity of each basic freeway, freeway weaving, freeway merge, and freeway diverge segment to be estimated. It is highly unlikely that every segment of a facility will have the same roadway, traffic, and control conditions and even less likely that they will have the same capacity.

Conceptual Approach to the Capacity of a Freeway Facility

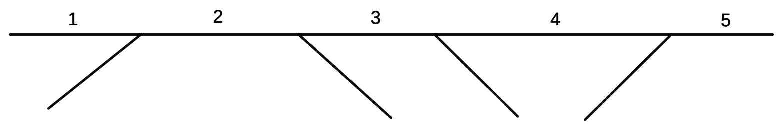
Consider the example shown in Exhibit 10-4. It illustrates five consecutive segments that are to be analyzed as one "freeway facility." Demand flow rates v_d, capacities c, and actual flow rates v_a are shown, as are the resulting v_d/c and v_a/c ratios. A lane is added in Segment 3 (even though this segment begins with an off-ramp), providing higher capacities for Segments 3, 4, and 5 than in Segments 1 and 2. The example analyzes three scenarios.

In Scenario 1, none of the demand flow rates exceeds the capacities of the segments that make up the facility. Thus, no breakdowns occur, and the actual flow rates are the same as the demand flow rates (i.e., $v_d = v_a$ for this scenario). None of the v_d/c or v_a/c ratios exceeds 1.00, although the highest ratios (0.978) occur in Segment 5.

Scenario 2 adds 200 vehicles per hour (veh/h) of demand to each segment (essentially another 200 veh/h of through freeway vehicles). In this case, Segment 5 will experience a breakdown—that is, the demand flow rate will exceed the capacity. In this segment, demand flow rate v_d differs from the actual flow rate v_a, as the actual flow rate v_a can never exceed the capacity c.

In Scenario 3, all demand flow rates are increased by 10%, which, in effect, keeps the relative values of the segment demand flow rates constant. In this case, demand flow rate will exceed capacity in Segments 4 and 5. Again, the demand flow rates and actual flow rates will differ in these segments.

Exhibit 10-4
Example of the Effect of Segment Capacity on a Freeway Facility



Scenario	Performance Measures	Freeway Segment 1	2	3	4	5
Scenario 1 (stable flow)	Demand v_d, veh/h	3,400	3,500	3,400	4,200	4,400
	Capacity c, veh/h	4,000	4,000	4,500	4,500	4,500
	Volume v_a, veh/h	3,400	3,500	3,400	4,200	4,400
	v_d/c ratio	0.850	0.875	0.756	0.933	0.978
	v_a/c ratio	0.850	0.875	0.756	0.933	0.978
Scenario 2 (add 200 veh/h to each segment)	Demand v_d, veh/h	3,600	3,700	3,600	4,400	4,600
	Capacity c, veh/h	4,000	4,000	4,500	4,500	4,500
	Volume v_a, veh/h	3,600	3,700	3,600	4,400	4,500
	v_d/c ratio	0.900	0.925	0.800	0.978	1.022
	v_a/c ratio	0.900	0.925	0.800	0.978	1.000
Scenario 3 (increase demand by 10% in all segments)	Demand v_d, veh/h	3,740	3,850	3,740	4,840	5,060
	Capacity c, veh/h	4,000	4,000	4,500	4,500	4,500
	Volume v_a, veh/h	3,740	3,850	3,740	4,500	4,500
	v_d/c ratio	0.935	0.963	0.831	1.078	1.120
	v_a/c ratio	0.935	0.963	0.831	1.000	1.000

Note: Shaded cells indicate segments where demand exceeds capacity.

This example highlights a number of points that make the analysis of freeway facilities very complicated:

1. It is critical to this methodology that the difference between demand flow rate v_d and actual flow rate v_a be highlighted and that both values be clearly and appropriately labeled.
2. In Scenarios 2 and 3, the analysis of Exhibit 10-4 is inadequate and misleading. In Scenario 2, when Segment 5 breaks down, queues begin to form and to propagate upstream. Thus, even though the demands in Segments 1 through 4 are less than the capacity of those segments, the queues generated by Segment 5 over time will propagate through Segments 1 through 4 and significantly affect their operation. In Scenario 3, Segments 4 and 5 fail, and queues are generated, which also propagate upstream over time.
3. It might be argued that the analysis of Scenario 1 is sufficient to understand the facility operation as long as all segments are undersaturated (i.e., all segment v_d/c ratios are less than or equal to 1.00). However, when any segment v_d/c ratio exceeds 1.00, such a simple analysis ignores the spreading impact of breakdowns in space and time.
4. In Scenarios 2 and 3, the segments downstream of Segment 5 will also be affected, as demand flow is prevented from reaching those segments by the Segment 5 (and Segment 4 in Scenario 3) breakdowns and queues.
5. In this example, it is also important to note that the segment(s) that break down first do not have the lowest capacities. Segments 1 and 2, with lower capacities, do not break down in any of the scenarios. Breakdown occurs first in Segment 5, which has one of the higher capacities.

Considering all these complications, the capacity of a freeway facility is defined as follows:

Freeway facility capacity is the capacity of the critical segment among those segments composing the defined facility. This capacity must, for analysis purposes, be compared with the demand flow rate on the critical segment.

The *critical segment* is defined as the segment that will break down first, given that all traffic, roadway, and control conditions do not change, including the spatial distribution of demands on each component segment. This definition is not a simple one. It depends on the relative demand characteristics and can change over time as the demand pattern changes. Facility capacity may be more than the capacity of the component segment with the lowest capacity. Therefore, it is important that individual segment demands and capacities be evaluated. The fact that one of these segments will be the critical one and will define the facility capacity does not diminish the importance of the capacities of other segments in the defined facility.

Base Capacity of Freeway Facilities

In the methodologies of Chapters 11, 12, and 13, a base capacity is used. The base capacity represents the capacity of the facility, assuming that there are no heavy vehicles in the traffic stream and that all drivers are regular users of the segment. The base capacity for all freeway segments varies with the free-flow speed, as indicated in Exhibit 10-5.

Exhibit 10-5
Free-Flow Speed vs. Base Capacity for Freeways

Free-Flow Speed (mi/h)	Base Capacity (pc/h/ln)
75	2,400
70	2,400
65	2,350
60	2,300
55	2,250

The equation given in Chapter 11, Basic Freeway Segments, for estimating the free-flow speed of a segment is as shown in Equation 10-1:

Equation 10-1

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\ TRD^{0.84}$$

where

FFS = free-flow speed (mi/h),

f_{LW} = adjustment for lane width (mi/h),

f_{LC} = adjustment for lateral clearance (mi/h), and

TRD = total ramp density (ramps/mi).

The process for determining the value of adjustment factors is described in Chapter 11.

Because the base capacity of a freeway segment is directly related to the free-flow speed, it is possible to construct a relationship between base capacity and the lane width, lateral clearance, and total ramp density of the segment. If the lane width and lateral clearance are taken to be their base values (12 and 6 ft, respectively), a relationship between base capacity and total ramp density emerges, as shown in Exhibit 10-6.

Base capacity is expressed as a flow rate for a 15-min analysis period, not a full-hour volume. It also represents a flow rate in pc/h, with no heavy vehicles, and a driver population familiar with the characteristics of the analysis segment.

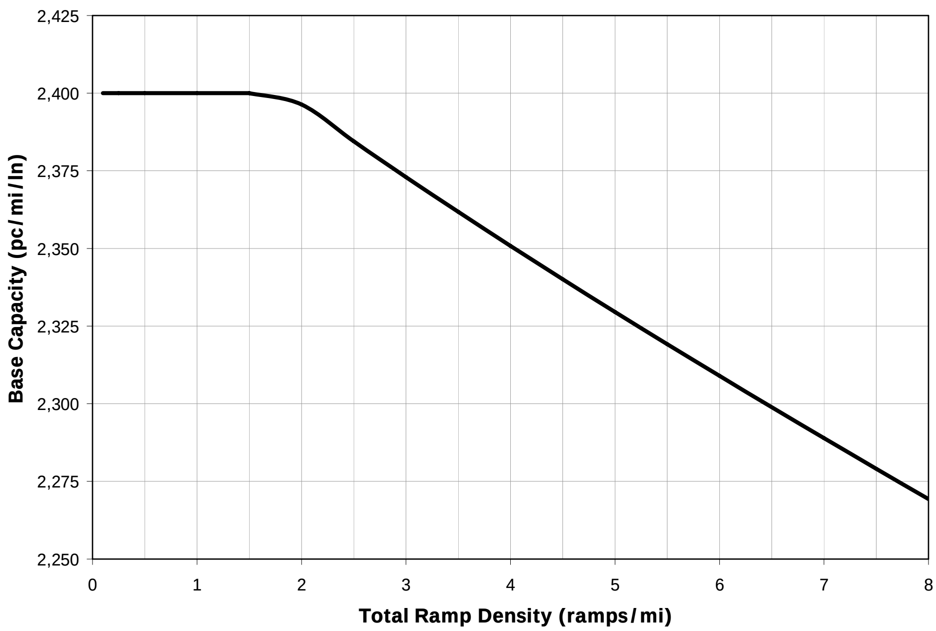


Exhibit 10-6
Base Capacity vs. Total Ramp Density

LIVE GRAPH
Click here to view

Segment Capacity vs. Facility Capacity

Free-flow speed is a characteristic of a length of freeway extending 3 mi upstream and 3 mi downstream of the center point of an analysis segment. The segment may be a basic freeway segment, a weaving segment, a merge segment, or a diverge segment. In essence, it is a measure of the impact of overall facility characteristics on the operation of the individual analysis segment centered in the defined 6-mi range.

This concept can be somewhat generalized where freeway facility analysis is involved. If conditions (particularly ramp density) are similar along a greater length of freeway, it is acceptable to compute the total ramp density for the greater length and apply it to all segments within the analysis length. This process assumes that moving the "center" of a 6-mi length for each component segment will not result in a significant change in the free-flow speed.

The capacity of a nearly homogeneous freeway facility is, for all practical purposes, the same as the capacity of a basic freeway segment with the same roadway and traffic characteristics. Consider the following:

- Merge and diverge segments have the same capacity as a similar basic freeway segment. As discussed in Chapter 13, the presence of merge and diverge segments on a freeway may affect operating characteristics, generally reducing speeds and increasing densities, but does not reduce capacity.
- Weaving segments often have per lane capacities that are less than those of the entering and leaving basic freeway segments. In almost all cases, however, weaving segments have more lanes than the entering and

leaving basic freeway segments. Thus, the impact on the capacity of the mainline freeway most often is negligible.

This does not mean, however, that the capacity of each component segment of a facility is the same. Each segment has its own demand and demand characteristics. Demand flow rate can change at every entry and exit point along the freeway, and the percent of heavy vehicles can change too. Terrain also can change at various points along the freeway.

Changes in heavy vehicle presence can change the capacity of individual segments within a defined facility. Changes in the split of movements in a weaving segment can change its capacity. In the same way, changes in the relative demand flows at on- and off-ramps can change the location of the critical segment within a defined facility and its capacity.

As noted previously, the capacity of a freeway facility is defined as the capacity of its critical segment.

LOS: COMPONENT SEGMENTS AND THE FREEWAY FACILITY

LOS of Component Segments

Chapters 11, 12, and 13 provide methodologies to determine the LOS in basic, weaving, merge, and diverge segments. In all cases, LOS F is identified when v_d/c is greater than 1.00. Such breakdowns are easily identified, and users are referred to this chapter.

This chapter's methodology provides an analysis of breakdown conditions, including the spatial and time impacts of a breakdown. Thus, in the performance of a facility-level analysis, LOS F in a component segment can be identified (*a*) when the segment v_d/c is greater than 1.00 and (*b*) when a queue from a downstream breakdown extends into an upstream segment. The latter cannot be done by using the individual segment analysis procedures of Chapters 11, 12, and 13.

Thus, when facility-level analysis is undertaken by using the methodology of this chapter, LOS F for a component segment will be identified in two different ways:

- When v_d/c is greater than 1.00, or
- When the density is greater than 45 pc/mi/ln for basic freeway segments or 43 pc/mi/ln for weaving, merge, or diverge segments.

The latter identifies segments in which queues have formed as a result of downstream breakdowns.

LOS for a Freeway Facility

Because LOS for basic, weaving, merge, and diverge segments on a freeway is defined in terms of density, LOS for a freeway facility is also defined on the basis of density.

A facility analysis will result in a density determination and LOS for each component segment. The facility LOS will be based on the weighted average density for all segments within the defined facility. Weighting is done on the

basis of segment length and the number of lanes in each segment, as shown in Equation 10-2:

Equation 10-2

$$D_F = \frac{\sum_{i=1}^{n} D_i \times L_i \times N_i}{\sum_{i=1}^{n} L_i \times N_i}$$

where

D_F = average density for the facility (pc/mi/ln),

D_i = density for segment i (pc/mi/ln),

L_i = length of segment i (ft),

N_i = number of lanes in segment i, and

n = number of segments in the defined facility.

The LOS criteria for a freeway facility are shown in Exhibit 10-7. They are the same criteria used for basic freeway segments.

Exhibit 10-7
LOS Criteria for Freeway Facilities

Level of Service	Density (pc/mi/ln)
A	≤11
B	>11–18
C	>18–26
D	>26–35
E	>35–45
F	>45 or any component v_d/c ratio > 1.00

Use of a LOS descriptor for the overall freeway facility must be done with care. It is critical that the LOS for individual segments composing the facility also be reported. Because the overall LOS is an average, it may mask serious problems in individual segments of the facility.

This is particularly important if one or more of the component segments are operating at LOS F. As described in this chapter's methodology section, the freeway facility methodology applies models to estimate the propagation of the effects of a breakdown in time and space. Where breakdowns exist in one or more segments of a facility, the average LOS is of limited use. The average LOS applies to a specific time period, usually 15 min.

While LOS A through D are defined by using the same densities that apply to basic freeway segments, LOS F for a facility is defined as a case in which any component segment of the freeway exceeds a v_d/c ratio of 1.00 or the average density over the defined facility exceeds 45 pc/mi/ln. In such a case, this chapter's methodology allows the analyst to map the impacts of this breakdown in time and space, and close attention to the individual LOS of component segments is necessary.

SERVICE FLOW RATES, SERVICE VOLUMES, AND DAILY SERVICE VOLUMES FOR A FREEWAY FACILITY

Just as each segment of a freeway facility has its own capacity, each segment also has a set of service flow rates SF_i for each LOS. A service flow rate is the maximum directional rate of flow that can be sustained in a given segment without violating the criteria for LOS *i*. Service flow rates are stated in vehicles per hour under prevailing roadway, traffic, and control conditions. By definition, the service flow rate for LOS E is synonymous with capacity for all uninterrupted-flow facilities and their component segments.

Chapters 11, 12, and 13 provide complete discussions of how to determine service flow rates for basic, weaving, merge, and diverge freeway segments.

A service volume SV_i is the maximum hourly directional volume that can be sustained in a given segment without violating the criteria for LOS *i* during the worst 15 min of the hour (period with the highest density) under prevailing roadway, traffic, and control conditions. Once a set of service flow rates has been established for a segment, the service volume is found from Equation 10-3:

Equation 10-3

$$SV_i = SF_i \times PHF$$

where

SV_i = service volume for LOS *i* (veh/h),

SF_i = service flow rate for LOS *i* (veh/h), and

PHF = peak hour factor.

A daily service volume DSV_i is the maximum total daily volume in both directions that can be sustained in a given segment without violating the criteria for LOS *i* in the peak direction in the worst 15 min of the peak hour under prevailing roadway, traffic, and control conditions. Given a set of service volumes for a segment, the daily service volume is found from Equation 10-4:

Equation 10-4

$$DSV_i = \frac{SV_i}{K \times D}$$

where

DSV_i = daily service volume (veh/day),

K = proportion of daily traffic occurring in the peak hour of the day, and

D = proportion of traffic in the peak direction during the peak hour of the day.

The capacity of a freeway facility has been defined as the capacity (under prevailing conditions) of the critical segment. For consistency, therefore, other service flow rates must also be applied to the critical segment.

For an overall understanding of the freeway facility, the LOS and service flow rates (or service volumes or daily service volumes) of the individual component segments must be considered along with the overall average LOS for the defined facility and its service flow rate.

GENERALIZED DAILY SERVICE VOLUMES FOR FREEWAY FACILITIES

Generalized daily service volume tables provide a means to assess all freeways in a region or jurisdiction quickly to determine which segments need to be assessed more carefully (using operational analysis) to ameliorate existing or pending problems.

To generate a generalized daily service volume table for freeway facilities, several simplifying assumptions must be made. The assumptions made here include the following:

1. All segments of the freeway have the same basic number of lanes (two, three, or four in each direction).
2. Lane widths are 12 ft, and lateral clearances are 6 ft.
3. All on-ramps and off-ramps handle the same percentage of freeway traffic. This setup maintains a reasonably consistent demand flow rate on each segment of the facility.
4. The first ramp on the defined freeway facility is an off-ramp. This assumption is necessary to implement Item 5, below.
5. Given the demand characteristics of Items 2 and 3, all daily service volumes are stated in terms of the demand *entering* the defined freeway facility at its upstream boundary.
6. The terrain is the same in all segments of the facility.
7. The heavy vehicle percentage is the same in all segments of the facility.

On the basis of these assumptions, generalized daily service-volume tables are shown in Exhibit 10-8 (for urban freeways) and Exhibit 10-9 (for rural freeways).

Generalized service volumes are provided for level and rolling terrain; for four-lane, six-lane, and eight-lane freeways (both directions); and for a variety of combinations of the *K*-factor and *D*-factor. To use the table, analysts must select a combination of *K* and *D* appropriate for their state or region. Additional assumptions made for urban and rural freeways are listed here.

Assumptions for urban freeways:

- Total ramp density = 3.00 ramps/mi (i.e., ⅓-mi average spacing between ramps);
- 5% trucks, no recreational vehicles (RVs), and no buses;
- *PHF* = 0.95; and
- f_p = 1.00.

Assumptions for rural freeways:

- Total ramp density = 0.20 ramp/mi (i.e., 5-mi average spacing between ramps);
- 12% trucks, no RVs, and no buses;
- *PHF* = 0.88; and
- f_p = 0.85.

Generalized daily service volumes are based on the maximum service flow rate values for basic freeway segments. Exhibit 11-17 (Chapter 11) shows maximum service flow rates *MSF* for basic freeway segments. They are converted to service flow rates under prevailing conditions by multiplying by the number of lanes in one direction *N*, the heavy-vehicle adjustment factor f_{HV}, and the driver-population adjustment factor f_p. Equation 10-3 and Equation 10-4 are then used to convert the service flow rate *SF* to a service volume *SV* and a daily service volume *DSV*.

By combining these equations, the daily service volumes *DSV* of Exhibit 10-8 and Exhibit 10-9 are estimated from Equation 10-5:

Equation 10-5

$$DSV_i = \frac{MSF_i \times N \times f_{HV} \times f_p \times PHF}{K \times D}$$

where all variables are as previously defined.

In applying Equation 10-5, the values of *MSF* are selected from Exhibit 11-17 (Chapter 11), and values for the heavy vehicle and driver population adjustment factors are computed in accordance with the methodology of Chapter 11. The *MSF* for LOS E, which is capacity, may be taken directly from Exhibit 10-5, based on the total ramp density, as lane widths and lateral clearances are standard and have no effect on the *FFS* and thus no effect on the resulting capacities.

Exhibit 10-8 and Exhibit 10-9 are provided for general planning use and should *not* be used to analyze any specific freeway or to make final decisions on important design features. A full operational analysis using this chapter's methodology is required for such specific applications.

The exhibits are useful, however, in evaluating the overall performance of many freeways within a jurisdiction, as a first pass in determining where problems might exist or arise, and in deciding where improvements might be needed. Any freeways identified as likely to experience problems or to need improvement, however, should be subjected to a full operational analysis before any detailed decisions on implementing specific improvements are made.

Daily service volumes are heavily affected by the *K*- and *D*-factors chosen as typical for the analysis. It is important that the analyst use values that are reasonable for the facilities under study. Also, if any characteristic differs significantly from the typical values used to develop Exhibit 10-8 and Exhibit 10-9, the values taken from these exhibits will not be representative of the study facilities.

Exhibit 10-8
Generalized Daily Service Volumes for Urban Freeway Facilities (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Freeways LOS B	LOS C	LOS D	LOS E	Six-Lane Freeways LOS B	LOS C	LOS D	LOS E	Eight-Lane Freeways LOS B	LOS C	LOS D	LOS E
Level Terrain													
0.08	0.50	54.2	75.5	94.1	108.9	81.3	113.3	141.1	163.4	108.4	151.1	188.1	217.8
	0.55	49.3	68.7	85.5	99.0	73.9	103.0	128.3	148.5	98.6	137.3	171.0	198.0
	0.60	45.2	62.9	78.4	90.8	67.8	94.4	117.6	136.1	90.4	125.9	156.8	181.5
	0.65	41.7	58.1	72.4	83.8	62.6	87.2	108.5	125.7	83.4	116.2	144.7	167.5
0.09	0.50	48.2	67.1	83.6	96.8	72.3	100.7	125.4	145.2	96.4	134.3	167.2	193.6
	0.55	43.8	61.0	76.0	88.0	65.7	91.6	114.0	132.0	87.6	122.1	152.0	176.0
	0.60	40.2	56.0	69.7	80.7	60.2	83.9	104.5	121.0	80.3	111.9	139.4	161.3
	0.65	37.1	51.6	64.3	74.5	55.6	77.5	96.5	111.7	74.1	103.3	128.6	148.9
0.10	0.50	43.4	60.4	75.3	87.1	65.1	90.6	112.9	130.7	86.8	120.9	150.5	174.2
	0.55	39.4	54.9	68.4	79.2	59.1	82.4	102.6	118.8	78.9	109.9	136.8	158.4
	0.60	36.1	50.4	62.7	72.6	54.2	75.5	94.1	108.9	72.3	100.7	125.4	145.2
	0.65	33.4	46.5	57.9	67.0	50.0	69.7	86.8	100.5	66.7	93.0	115.8	134.0
0.11	0.50	39.4	54.9	68.4	79.2	59.1	82.4	102.6	118.8	78.9	109.9	136.8	158.4
	0.55	35.8	49.9	62.2	72.0	53.8	74.9	93.3	108.0	71.7	99.9	124.4	144.0
	0.60	32.9	45.8	57.0	66.0	49.3	68.7	85.5	99.0	65.7	91.6	114.0	132.0
	0.65	30.3	42.3	52.6	60.9	45.5	63.4	78.9	91.4	60.7	84.5	105.3	121.8
Rolling Terrain													
0.08	0.50	51.7	72.0	89.7	103.8	77.5	108.0	134.5	155.8	103.4	144.0	179.4	207.7
	0.55	47.0	65.5	81.5	94.4	70.5	98.2	122.3	141.6	94.0	131.0	163.1	188.8
	0.60	43.1	60.0	74.7	86.5	64.6	90.0	112.1	129.8	86.2	120.0	149.5	173.1
	0.65	39.8	55.4	69.0	79.9	59.7	83.1	103.5	119.8	79.5	110.8	138.0	159.7
0.09	0.50	46.0	64.0	79.7	92.3	68.9	96.0	119.6	138.4	91.9	128.0	159.5	184.6
	0.55	41.8	58.2	72.5	83.9	62.7	87.3	108.7	125.9	83.6	116.4	145.0	167.8
	0.60	38.3	53.4	66.4	76.9	57.4	80.0	99.7	115.4	76.6	106.7	132.9	153.8
	0.65	35.3	49.2	61.3	71.0	53.0	73.9	92.0	106.5	70.7	98.5	122.7	142.0
0.10	0.50	41.4	57.6	71.8	83.1	62.0	86.4	107.6	124.6	82.7	115.2	143.5	166.1
	0.55	37.6	52.4	65.2	75.5	56.4	78.6	97.9	113.3	75.2	104.8	130.5	151.0
	0.60	34.5	48.0	59.8	69.2	51.7	72.0	89.7	103.8	68.9	96.0	119.6	138.4
	0.65	31.8	44.3	55.2	63.9	47.7	66.5	82.8	95.8	63.6	88.6	110.4	127.8
0.11	0.50	37.6	52.4	65.2	75.5	56.4	78.6	97.9	113.3	75.2	104.8	130.5	151.0
	0.55	34.2	47.6	59.3	68.7	51.3	71.4	89.0	103.0	68.4	95.2	118.6	137.3
	0.60	31.3	43.7	54.4	62.9	47.0	65.5	81.5	94.4	62.7	87.3	108.7	125.9
	0.65	28.9	40.3	50.2	58.1	43.4	60.4	75.3	87.1	57.8	80.6	100.4	116.2

Note: Assumptions include the following: 5% trucks, 0% buses, 0% RVs, 0.95 *PHF*, 3 ramps/mi, f_p = 1.00, 12-ft lanes, and 6-ft lateral clearance. Values do not represent specific segment characteristics.

Exhibit 10-9
Generalized Daily Service Volumes for Rural Freeway Facilities (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Freeways LOS B	LOS C	LOS D	LOS E	Six-Lane Freeways LOS B	LOS C	LOS D	LOS E	Eight-Lane Freeways LOS B	LOS C	LOS D	LOS E
Level Terrain													
0.09	0.50	41.1	54.9	66.2	75.3	61.6	82.3	99.3	112.9	82.2	109.8	132.4	150.5
	0.55	37.4	49.9	60.2	68.4	56.0	74.8	90.2	102.6	74.7	99.8	120.3	136.9
	0.60	34.2	45.7	55.1	62.7	51.4	68.6	82.7	94.1	68.5	91.5	110.3	125.5
	0.65	31.6	42.2	50.9	57.9	47.4	63.3	76.4	86.9	63.2	84.4	101.8	115.8
0.10	0.50	37.0	49.4	59.6	67.7	55.5	74.1	89.3	101.6	74.0	98.8	119.1	135.5
	0.55	33.6	44.9	54.1	61.6	50.4	67.4	81.2	92.4	67.2	89.8	108.3	123.2
	0.60	30.8	41.2	49.6	56.5	46.2	61.7	74.4	84.7	61.6	82.3	99.3	112.9
	0.65	28.4	38.0	45.8	52.1	42.7	57.0	68.7	78.2	56.9	76.0	91.6	104.2
0.11	0.50	33.6	44.9	54.1	61.6	50.4	67.4	81.2	92.4	67.2	89.8	108.3	123.2
	0.55	30.6	40.8	49.2	56.0	45.8	61.2	73.8	84.0	61.1	81.6	98.4	112.0
	0.60	28.0	37.4	45.1	51.3	42.0	56.1	67.7	77.0	56.0	74.8	90.2	102.6
	0.65	25.9	34.5	41.6	47.4	38.8	51.8	62.5	71.1	51.7	69.1	83.3	94.7
0.12	0.50	30.8	41.2	49.6	56.5	46.2	61.7	74.4	84.7	61.6	82.3	99.3	112.9
	0.55	28.0	37.4	45.1	51.3	42.0	56.1	67.7	77.0	56.0	74.8	90.2	102.6
	0.60	25.7	34.3	41.4	47.0	38.5	51.5	62.0	70.6	51.4	68.6	82.7	94.1
	0.65	23.7	31.7	38.2	43.4	35.6	47.5	57.3	65.1	47.4	63.3	76.4	86.9
Rolling Terrain													
0.09	0.50	36.9	49.3	59.4	67.6	55.4	74.0	89.2	101.4	73.8	98.6	118.9	135.2
	0.55	33.6	44.8	54.0	61.5	50.3	67.2	81.1	92.2	67.1	89.6	108.1	122.9
	0.60	30.8	41.1	49.5	56.3	46.1	61.6	74.3	84.5	61.5	82.2	99.1	112.7
	0.65	28.4	37.9	45.7	52.0	42.6	56.9	68.6	78.0	56.8	75.9	91.5	104.0
0.10	0.50	33.2	44.4	53.5	60.9	49.8	66.6	80.3	91.3	66.4	88.7	107.0	121.7
	0.55	30.2	40.3	48.6	55.3	45.3	60.5	73.0	83.0	60.4	80.7	97.3	110.6
	0.60	27.7	37.0	44.6	50.7	41.5	55.5	66.9	76.1	55.4	74.0	89.2	101.4
	0.65	25.6	34.1	41.2	46.8	38.3	51.2	61.7	70.2	51.1	68.3	82.3	93.6
0.11	0.50	30.2	40.3	48.6	55.3	45.3	60.5	73.0	83.0	60.4	80.7	97.3	110.6
	0.55	27.5	36.7	44.2	50.3	41.2	55.0	66.3	75.4	54.9	73.3	88.4	100.6
	0.60	25.2	33.6	40.5	46.1	37.7	50.4	60.8	69.2	50.3	67.2	81.1	92.2
	0.65	23.2	31.0	37.4	42.6	34.8	46.5	56.1	63.8	46.5	62.1	74.8	85.1
0.12	0.50	27.7	37.0	44.6	50.7	41.5	55.5	66.9	76.1	55.4	74.0	89.2	101.4
	0.55	25.2	33.6	40.5	46.1	37.7	50.4	60.8	69.2	50.3	67.2	81.1	92.2
	0.60	23.1	30.8	37.2	42.3	34.6	46.2	55.7	63.4	46.1	61.6	74.3	84.5
	0.65	21.3	28.4	34.3	39.0	31.9	42.7	51.4	58.5	42.6	56.9	68.6	78.0

Note: Assumptions include the following: 12% trucks, 0% buses, 0% RVs, 0.88 *PHF*, 0.2 ramp/mi, f_p = 0.85, 12-ft lanes, and 6-ft lateral clearance. Values do not represent specific segment characteristics.

ACTIVE TRAFFIC MANAGEMENT AND OTHER MEASURES TO IMPROVE PERFORMANCE

Active traffic management (ATM) consists of the dynamic and continuous monitoring and control of traffic operations on a facility to improve its performance. Examples of ATM measures include congestion pricing, ramp metering, changeable message signs, incident response programs, and speed harmonization (variable speed limits).

ATM measures can influence both the nature of demand for the facility and the ability of the facility to deliver the capacity tailored to serve the demand. ATM measures can improve facility performance, sometimes significantly.

Other advanced design and management measures, not specifically included in the definition of ATM, can also significantly improve facility performance. These measures include auxiliary lanes, narrow lanes, high-occupancy vehicle (HOV) lanes, temporary use of shoulders, and designated truck lanes and ramps.

This methodology does not reflect all these measures. However, ramp metering can be taken into account by altering on-ramp demands in accordance

with metering rates. Auxiliary lanes and narrow lanes are taken into account in the segment methodologies for basic freeway segments and weaving segments.

Other measures are not accounted for in this methodology. Chapter 35 provides a more detailed discussion of ATM and other advanced design and management strategies and insight into how their impacts may be evaluated.

2. METHODOLOGY

The methodology presented in this chapter provides for the integrated analysis of a freeway facility composed of connected segments. The methodology builds on the models and procedures for individual segments, as described in Chapter 11, Basic Freeway Segments; Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments.

SCOPE OF THE METHODOLOGY

Because the freeway facility methodology builds on the segment methodologies of Chapters 11, 12, and 13, it incorporates all aspects of those chapters' methodologies. This methodology adds the ability to consider a number of linked segments over a number of time periods and to determine some overall operational parameters that allow for the assessment of a facility LOS and capacity.

This methodology also adds the ability to analyze operations when LOS F exists on one or more segments of the defined facility. In Chapters 11, 12, and 13, the existence of a breakdown (LOS F) is identified for a given segment, as appropriate. The segment methodologies do not, however, provide tools for analyzing the impacts of such breakdowns over time and space.

The methodology analyzes a set of connected segments over a set of sequential 15-min periods. In deciding which segments and time periods to analyze, two principles should be observed:

1. The first and last segments of the defined facility should *not* operate at LOS F.
2. The first and last time periods of the analysis should *not* include any segments that operate at LOS F.

When the first segment operates at LOS F, there is a queue extending upstream that is not included in the facility definition and that therefore cannot be analyzed. When the last segment operates at LOS F, there may be a downstream bottleneck outside the facility definition. Again, the impacts of this congestion cannot be evaluated when it is not fully contained within the defined facility. LOS F in either the first or last time period creates similar problems with regard to time. If the first time period is at LOS F, then LOS F may exist in previous time periods as well. If the last time period is at LOS F, subsequent periods may be at LOS F as well. The impacts of a breakdown cannot be fully analyzed unless it is fully contained within the defined facility and defined total analysis period. The same problems would exist if the analysis were conducted by using simulation.

There is no limit to the number of time periods that can be analyzed. The length of the freeway should be less than the distance a vehicle traveling at the average speed can achieve in 15 min. This specification generally results in a maximum facility length between 9 and 12 mi.

This methodology is based on research sponsored by the Federal Highway Administration (*1*).

LIMITATIONS OF THE METHODOLOGY

The methodology has the following limitations:

1. The methodology does not account for the delays caused by vehicles using alternative routes or vehicles leaving before or after the analysis period.
2. Multiple overlapping breakdowns or bottlenecks are difficult to analyze and cannot be fully evaluated by this methodology. Other tools may be more appropriate for specific applications beyond the capabilities of the methodology. Consult Chapter 6, HCM and Alternative Analysis Tools, for a discussion of simulation and other models.
3. Spatial, temporal, modal, and total demand responses to traffic management strategies are not automatically incorporated into the methodology. On viewing the facility traffic performance results, the analyst can modify the demand input manually to analyze the effect of user-demand responses and traffic growth. The accuracy of the results depends on the accuracy of the estimation of user-demand responses.
4. The methodology can address local oversaturated flow but cannot directly address systemwide oversaturation flow conditions.
5. The completeness of the analysis will be limited if freeway segments in the first time interval, the last time interval, and the first freeway segment (in all time periods) have demand-to-capacity ratios greater than 1.00. The rationale for these limitations is discussed in the section on demand-to-capacity ratio.
6. The existence of HOV lanes on freeways raises the issues of the operating characteristics of such lanes and their effect on operating characteristics on the remainder of the freeway. The methodology does not directly address separated HOV facilities and does not account for the interactions between HOV lanes and mixed-flow lanes and the weaving that may be produced.
7. The method does not address conditions in which off-ramp capacity limitations result in queues that extend onto the freeway or affect the behavior of off-ramp vehicles.
8. The method does not address toll plaza operations or their effect on freeway facility operations.

Given enough time, the analyst can analyze a completely undersaturated time–space domain manually, although it is very difficult and time-consuming. It is not expected that analysts will ever manually analyze a time–space domain that includes oversaturation. FREEVAL-2010 is a computational engine that can be used to implement the methodology, regardless of whether the time–space domain contains oversaturated segments and time periods. It is available in the Technical Reference Library section of Volume 4 of the *Highway Capacity Manual* (HCM).

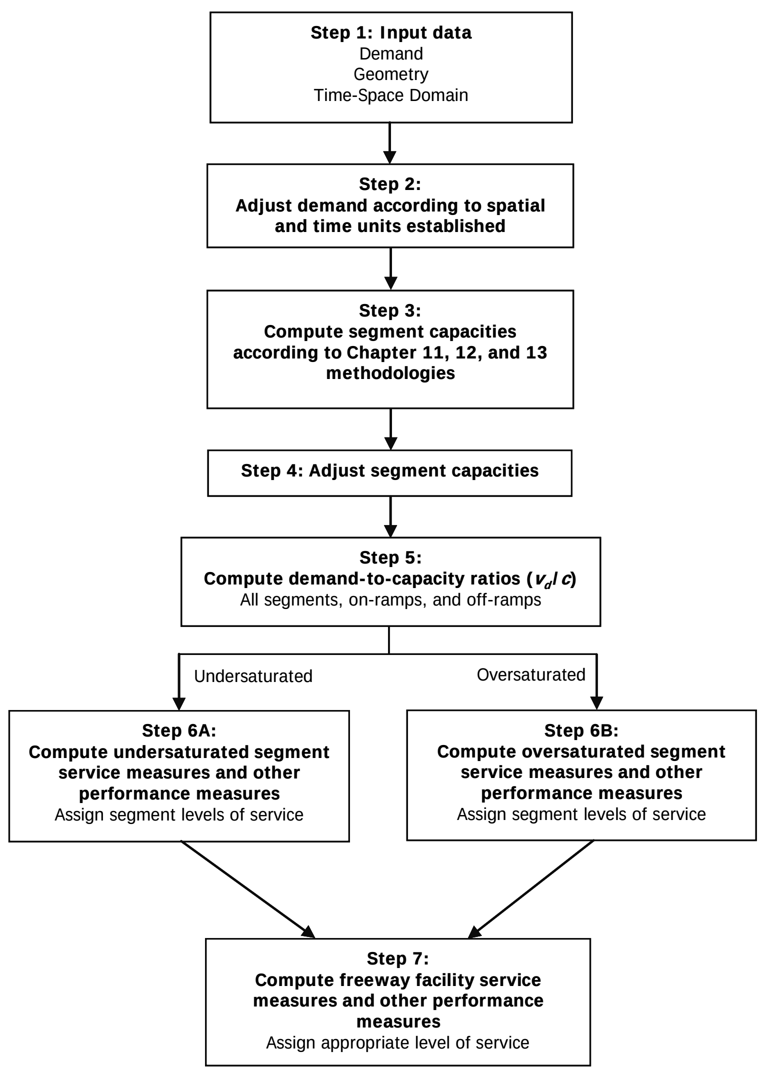
Because this chapter's methodology incorporates the methodologies for basic, weaving, merging, and diverging freeway segments, the limitations of those procedures also apply here.

The method does not include analysis of the street-side terminals of freeway on- and off-ramps. The methodologies of Chapters 18, 19, 20, and 21 should be used for intersections that are signalized, two-way STOP-controlled, all-way STOP-controlled, and roundabouts, respectively. Chapter 22, Interchange Ramp Terminals, provides a more comprehensive analysis of freeway interchanges where the street-side ramp terminals are signalized intersections or roundabouts.

OVERVIEW

Exhibit 10-10 summarizes the methodology for analyzing freeway facilities. The methodology adjusts vehicle speeds appropriately to account for the effects in adjacent segments. The methodology can analyze freeway traffic management strategies only in cases for which 15-min intervals are appropriate and for which reliable data for estimated capacity and demand exist.

Exhibit 10-10
Freeway Facility
Methodology



COMPUTATIONAL STEPS

The purpose of this section is to describe the methodology's computational modules. To simplify the presentation, the focus is on the function of, and rationale for, each module. Chapter 25 presents an expanded version of this section, including all the supporting analytical models and equations.

Step 1: Input Data

Data concerning demand, geometry, and the time–space domain must be specified. As the methodology builds on segment analysis, all data for each segment and each time period must be provided, as indicated in Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments.

Demand

Demand flow rates must be specified for each segment and time period. Because analysis of multiple time periods is based on consecutive 15-min periods, the demand flow rates for each period must be provided. This condition is in addition to the requirements for isolated segment analyses.

Demand flow rates must be specified for the entering freeway mainline flow and for each on-ramp and off-ramp within the defined facility. The following information is needed for each time period to determine the demand flow rate:

- Demand flow rate (veh/h),
- Percent trucks (%),
- Percent RVs (%), and
- Driver population factor (f_p).

For weaving segments, demand flow rates must be identified by component movement: freeway to freeway, ramp to freeway, freeway to ramp, and ramp to ramp. Where this level of detail is not available, the following procedure may be used to estimate the component flows. It is not recommended, however, as weaving segment performance is sensitive to the split of demand flows.

- *Ramp-weave segments:* Assume that the ramp-to-ramp flow is 0. The ramp-to-freeway flow is then equal to the on-ramp flow; the freeway-to-ramp flow is then equal to the off-ramp flow.
- *Major weave segments:* On-ramp flow is apportioned to the two exit legs (freeway and ramp) in the same proportion as the total flow on the exit legs (freeway and ramp).

The driver population factor is normally 1.00, unless the driver population is dominated by unfamiliar users, in which case a value between 0.85 and 1.00 is assigned, on the basis of local characteristics and knowledge.

Geometry

All geometric features for each segment of the facility must be specified, including the following:

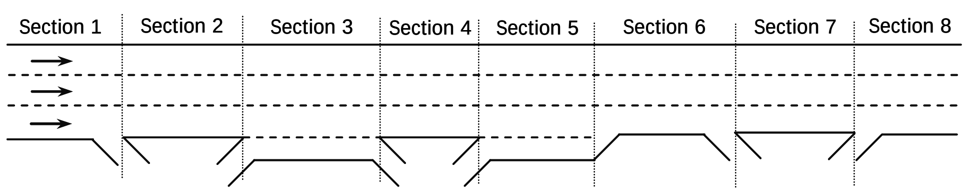
- Number of lanes;
- Average lane width;
- Right-side lateral clearance;
- Terrain;
- Free-flow speed; and
- Location of merge, diverge, and weaving segments, with all internal geometry specified, including the number of lanes on ramps and at ramp–freeway junctions or within weaving segments, lane widths, existence and length of acceleration or deceleration lanes, distances between merge and diverge points, and the details of lane configuration where relevant.

Geometry does not change by time period, so this information is given only once, regardless of the number of time periods under study.

Time–Space Domain

A time–space domain for the analysis must be established. The domain consists of a specification of the freeway *sections* included in the defined facility and an identification of the time intervals for which the analysis is to be conducted. A typical time–space domain is shown in Exhibit 10-11.

Exhibit 10-11
Example Time–Space Domain for Freeway Facility Analysis



Time Step	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Section 7	Section 8
1								
2								
3								
4								
5								
6								
7								
8								

The horizontal scale indicates the distance along the freeway facility. A freeway *section* boundary occurs where there is a change in demand—that is, at each on-ramp or off-ramp or where a lane is added or dropped. These areas are referred to as *sections*, because adjustments will be made within the procedure to determine where *segment* boundaries should be for analysis. This process relies on the influence areas of merge, diverge, and weaving segments, discussed earlier in this chapter, and on variable length limitations specified in Chapter 12 for weaving segments and in Chapter 13 for merge and diverge segments.

The vertical scale indicates the study time duration. Time extends down the time–space domain, and the scale is divided into 15-min intervals. In the example shown, there are 8 sections and 8 time steps, yielding 8 × 8 = 64 time–space cells, each of which will be analyzed within the methodology.

The boundary conditions of the time–space domain are extremely important. The time–space domain will be analyzed as an independent freeway facility having no interactions with upstream or downstream portions of the freeway, or any connecting facilities, including other freeways and surface facilities. Therefore, no congestion should occur along the four boundaries of the time–space domain. The cells located along the four boundaries should all have demands less than capacity and should contain undersaturated flow conditions. A proper analysis of congestion within the time–space domain can occur only if the congestion is limited to internal cells not along the time–space boundaries.

Converting the Horizontal Scale from Sections to Analysis Segments

The sections of the defined freeway facility are established by using points where demand changes or where lanes are added or subtracted. This, however, does *not* fully describe individual *segments* for analysis within the methodology. The conversion from sections to analysis segments can be done manually by applying the principles discussed here.

Chapter 13, Freeway Merge and Diverge Segments, indicates that each merge segment extends from the merge point to a point 1,500 ft downstream of it. Each diverge segment extends from the diverge point to a point 1,500 ft upstream of it. This allows for a number of scenarios affecting the definition of analysis segments within the defined freeway.

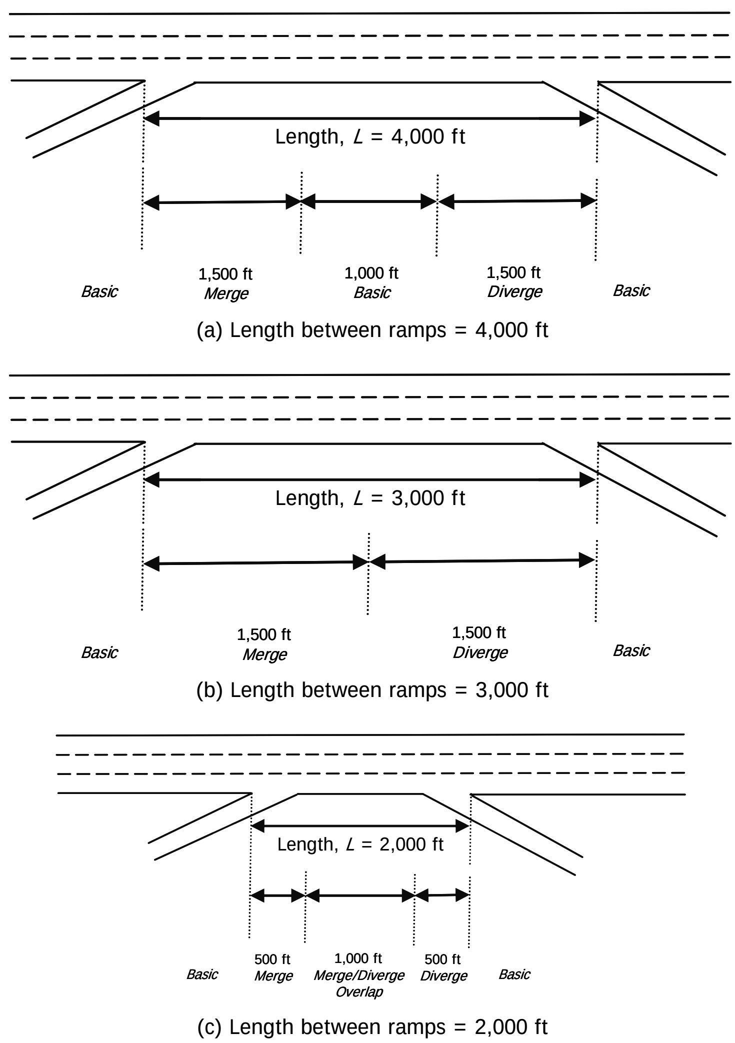
Consider the illustration of Exhibit 10-12. It shows a one-lane on-ramp followed by a one-lane off-ramp with no auxiliary lane between them. The illustration assumes that there are no upstream or downstream ramps or weaving segments that impinge on this section.

In Exhibit 10-12(a), there are 4,000 ft between the two ramps. Therefore, the merge segment extends 1,500 ft downstream, and the diverge segment extends 1,500 ft upstream, which leaves a 1,000-ft basic freeway segment between them.

In Exhibit 10-12(b), there are 3,000 ft between the two ramps. The two 1,500-ft ramp influence areas define the entire length. Therefore, there is no basic freeway segment between the merge and diverge segments.

In Exhibit 10-12(c), the situation is more complicated. With only 2,000 ft between the ramps, the merge and diverge influence areas overlap for a distance of 1,000 ft.

Exhibit 10-12
Defining Analysis Segments for a Ramp Configuration

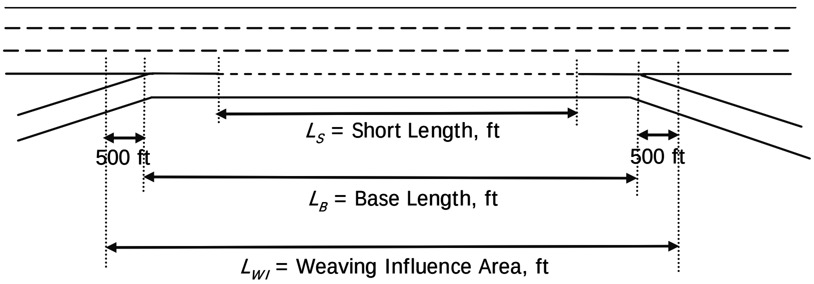


Chapter 13, Freeway Merge and Diverge Segments, covers this situation. Where ramp influence areas overlap, the analysis is conducted for each ramp separately. The analysis producing the worst LOS (or service measure value if the LOS is equivalent) is used to define operations in the overlap area.

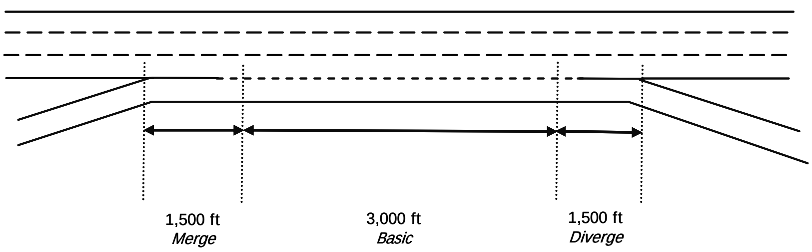
The facility methodology goes through the logic of distances and segment definitions to convert section boundaries to segment boundaries for analysis. If the distance between an on-ramp and off-ramp is less than the full influence area of 1,500 ft, the worst case is applied to the distance between the ramps, while basic segment criteria are applied to segments upstream of the on-ramp and downstream of the off-ramp.

A similar situation can arise where weaving configurations exist. Exhibit 10-13 illustrates a weaving configuration within a defined freeway facility. In this case, the distance between the merge and diverge ends of the configuration must be compared with the maximum length of a weaving segment, L_{wMAX}. If the distance between the merge and diverge points is less than or equal to L_{wMAX}, then the entire segment is analyzed as a weaving segment, as shown in Exhibit 10-13(a).

Exhibit 10-13
Defining Analysis Segments for a Weaving Configuration



(a) Case I: $L_B \leq L_{wMAX}$ (weaving segment exists)



(b) Case II: $L_B > L_{wMAX}$ (isolated merge and diverge exists)

Three lengths are involved in analyzing a weaving segment:

- The base length of the segment, measured from the points where the edges of the travel lanes of the merging and diverging roadways converge (L_B);
- The influence area of the weaving segment (L_{WI}), which includes 500 ft upstream and downstream of L_B; and
- The short length of the segment, defined as the distance over which lane changing is not prohibited or dissuaded by markings (L_S).

The latter is the length that is used in all the predictive models for weaving segment analysis. The results of these models, however, apply to a distance of L_B + 500 ft upstream and L_B + 500 ft downstream. For further discussion of the various lengths applied to weaving segments, consult Chapter 12.

If the distance between the merge and diverge points is greater than L_{wMAX}, then the merge and diverge segments are too far apart to form a weaving segment. As shown in Exhibit 10-13(b), the merge and diverge segments are treated separately, and any distance remaining between the merge and diverge influence areas is treated as a basic freeway segment.

In the Chapter 12 weaving methodology, the value of L_{wMAX} depends on a number of factors, including the split of component flows, demand flows, and other traffic factors. A weaving configuration could therefore qualify as a weaving segment in some analysis periods and as separate merge, diverge, and possibly basic segments in others.

In segmenting the freeway facility for analysis, merge, diverge, and weaving segments are identified as illustrated in Exhibit 10-12 and Exhibit 10-13. All segments not qualifying as merge, diverge, or weaving segments are basic freeway segments.

However, a long basic freeway section may have to be divided into multiple segments. This situation occurs when there is a sharp break in terrain within the section. For example, a 5-mi section may have a constant demand and a constant number of lanes. If there is a 2-mi level terrain portion followed by a 4% grade that is 3 mi long, then the level terrain portion and the specific grade portion would be established as two separate, consecutive basic freeway segments.

Step 2: Adjust Demand According to Spatial and Time Units Established

Traffic counts taken at each entrance to and exit from the defined freeway facility (including the mainline entrance and mainline exit) for each time interval serve as inputs to the methodology. While entrance counts are considered to represent the current entrance demands for the freeway facility (provided that there is not a queue on the freeway entrance), the exit counts may not represent the current exit demands for the freeway facility because of congestion within the defined facility.

For planning applications, estimated traffic demands at each entrance to and exit from the freeway facility for each time interval serve as input to the methodology. The sum of the input demands must equal the sum of the output demands in every time interval.

Once the entrance and exit demands are calculated, the demands for each cell in every time interval can be estimated. The segment demands can be thought of as filtering across the time–space domain and filling each cell of the time–space matrix.

Demand estimation is needed if the methodology uses actual freeway counts. If demand flows are known or can be projected, they are used directly without modification.

The methodology includes a demand estimation model that converts the input set of freeway exit 15-min counts to a set of vehicle flows that desire to exit the freeway in a given 15-min period. This demand may not be the same as the 15-min exit count because of upstream congestion within the defined freeway facility.

The procedure sums the freeway entrance demands along the entire directional freeway facility, including the entering mainline segment, and compares this sum with the sum of freeway exit counts along the directional freeway facility, including the departing mainline segment. This procedure is repeated for each time interval. The ratio of the total facility entrance counts to total facility exit counts is called the *time interval scale factor* and should approach 1.00 when the freeway exit counts are, in fact, freeway exit demands.

Scale factors greater than 1.00 indicate increasing levels of congestion within the freeway facility, with exit counts underestimating the actual freeway exit demands. To provide an estimate of freeway exit demand, each freeway exit count is multiplied by the time interval scale factor.

Equation 10-6 and Equation 10-7 summarize this process.

$$f_{TISi} = \frac{\sum_j V_{ON15ij}}{\sum_j V_{OFF15ij}}$$

Equation 10-6

$$V_{dOFF15ij} = V_{OFF15ij} \times f_{TISi}$$

Equation 10-7

where

f_{TISi} = time-interval scale factor for time period *i*,

V_{ON15ij} = 15-min entering count for time period *i* and entering location *j* (veh),

$V_{OFF15ij}$ = 15-min exit count for time period *i* and exiting location *j* (veh), and

$V_{dOFF15ij}$ = adjusted 15-min exit demand for time period *i* and exiting location *j* (veh).

Once the entrance and exit demands are determined, the traffic demands for each section and each time period can be calculated. On the time–space domain, section demands can be viewed as projecting horizontally across Exhibit 10-11, with each cell containing an estimate of its 15-min demand.

Because each time period is separately balanced, it is advisable to limit the total length of the defined facility to a distance that can be traversed within 15 min. In practical terms, this practice limits the length of the facility to 9 to 12 mi.

Step 3: Compute Segment Capacities According to Chapter 11, 12, and 13 Methodologies

Segment capacity estimates are determined by the methodologies of Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments. All estimates of segment capacity should be carefully reviewed and compared with local knowledge and available traffic information for the study site, particularly where known bottlenecks exist.

On-ramp and off-ramp roadway capacities are also determined in this step with the Chapter 13 methodology. On-ramp demands may exceed on-ramp capacities and limit the traffic demand entering the facility. Off-ramp demands may exceed off-ramp capacities and cause congestion on the freeway, although that impact is not accounted for in this methodology.

All capacity results are stated in vehicles per hour under prevailing roadway and traffic conditions.

The effect of a predetermined ramp-metering plan can be evaluated in this methodology by overriding the computed ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to reflect the specified ramp-metering rate. This feature not only allows for evaluating a prescribed ramp-metering plan but also permits the user to improve the ramp-metering plan through experimentation.

Freeway design improvements can be evaluated with this methodology by modifying the design features of any portion of the freeway facility. For example, the effects of adding auxiliary lanes at critical locations and full lanes over multiple segments can be assessed.

Step 4: Adjust Segment Capacities

Segment capacities can be affected by a number of conditions not normally accounted for in the segment methodologies of Chapters 11, 12, and 13. These reductions include the effects of short-term and long-term lane closures for construction or major maintenance operations, the effects of adverse weather conditions, and the effects of other environmental factors.

At lane drops, permanent reductions in capacity occur. They are included in the base methodology, which automatically accounts for the capacity of segments on the basis of the number of lanes in the segment and other prevailing conditions.

Capacity Reductions due to Construction and Major Maintenance Operations

Capacity reductions due to construction activities can be divided into short-term work-zone lane closures, typically for maintenance, and long-term lane closures, typically for construction. A primary distinction between short-term work zones and long-term construction zones is the nature of the barriers used to demarcate the work area. Long-term construction zones generally use portable concrete barriers, while short-term work zones use standard channeling devices (e.g., traffic cones, drums) in accordance with the *Manual on Uniform Traffic Control Devices for Streets and Highways* (2). Capacity reductions due to long-term construction or major maintenance operations generally last several weeks, months, or even years, depending on the nature of the work. Short-term closures generally last a few hours.

Short-Term Work Zones

Research (*3*) suggests that a capacity of 1,600 pc/h/ln be used for short-term freeway work zones, regardless of the lane-closure configuration. However, for some types of closures, a higher value could be appropriate.

This base value should be adjusted for other conditions, as follows:

1. *Intensity of work activity:* The intensity of work activity refers to the number of workers on the site, the number and size of work vehicles in use, and the proximity of the work activity to the travel lanes. Unusual types of work also contribute to intensity in terms of rubbernecking by drivers passing through the site. Research (*3*) suggests that the base value of 1,600 pc/h/ln be adjusted by as much as ±10% for work activity that is more or less intensive than normal. It does not, however, define what constitutes "normal" intensity, so this factor should be applied on the basis of professional judgment and local experience.

6. *Effects of heavy vehicles:* Because the base value is given in terms of pc/h/ln, it is recommended that the heavy vehicle adjustment factor (f_{HV}) be applied. A complete discussion of the heavy vehicle adjustment factor and its determination are included in Chapter 11, Basic Freeway Segments. Equation 10-8 shows how the factor is determined.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Equation 10-8

where

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of trucks and buses in the traffic stream,

P_R = proportion of RVs in the traffic stream,

E_T = passenger-car equivalent for trucks and buses, and

E_R = passenger-car equivalent for RVs.

Passenger-car equivalents for trucks and buses and for RVs may be found in Chapter 11, Basic Freeway Segments.

7. *Presence of ramps:* If there is an entrance ramp within the taper area approaching the lane closure or within 500 ft downstream of the beginning of the full lane closure, the ramp will have a noticeable effect on the capacity of the work zone for handling mainline traffic. This situation arises in two ways: (*a*) the ramp traffic generally forces its way in, so it directly reduces the amount of mainline traffic that can be handled, and (*b*) the added turbulence in the merge area may slightly reduce capacity (even though such turbulence does *not* reduce capacity on a normal freeway segment without lane closures). If at all possible, on-ramps should be located at least 1,500 ft upstream of the beginning of the full lane closure to maximize the total work zone throughput. If that cannot be done, then either the ramp volume should be added to the mainline volume to be served or the capacity of the work zone should be decreased by the ramp volume (up to a maximum of one-half of the capacity of one lane) on the assumption that, at very high volumes, mainline and ramp vehicles will alternate.

Equation 10-9 is used to estimate the resulting reduced capacity in vehicles per hour.

$$c_a = \{[(1{,}600 + I) \times f_{HV}] \times N\} - R$$

Equation 10-9

where

c_a = adjusted mainline capacity (veh/h);

I = adjustment factor for type, intensity, and proximity of work activity, pc/h/ln (ranges between ±160 pc/h/ln);

f_{HV} = heavy-vehicle adjustment factor;

N = number of lanes open through the work zone; and

R = manual adjustment for on-ramps (veh/h).

Long-Term Construction Zones

There have been many studies of long-term construction zone capacities. They are summarized in Exhibit 10-14.

Exhibit 10-14
Capacity of Long-Term Construction Zones (veh/h/ln)

State	Normal Lanes to Reduced Lanes: 2 to 1	3 to 2	3 to 1	4 to 3	4 to 2	4 to 1	Source
TX	1,340		1,170				(*4*)
NC	1,690		1,640				(*5*)
CT	1,500–1,800		1,500–1,800				(*6*)
MO	1,240	1,430	960	1,480	1,420		(*7*)
NV	1,375–1,400		1,375–1,400				(*6*)
OR	1,400–1,600		1,400–1,600				(*6*)
SC	950		950				(*6*)
WA	1,350		1,450				(*6*)
WI	1,560–1,900		1,600–2,000		1,800–2,100		(*6*, *8*)
FL	1,800		1,800				(*9*)
VA	1,300	1,300	1,300	1,300	1,300	1,300	(*10*)
IA	1,400–1,600	1,400–1,600	1,400–1,600	1,400–1,600	1,400–1,600	1,400–1,600	(*11*)
MA	1,340	1,490	1,170	1,520	1,480	1,170	(*12*)
Default	**1,400**	**1,450**	**1,450**	**1,500**	**1,450**	**1,350**	

Source: Adapted from Chatterjee et al. (*13*).

It is easy to see from Exhibit 10-14 that capacities through long-term construction zones are highly variable and depend on many site-specific characteristics. Therefore, it is better to base this adjustment on local data and experience. If such data do not exist and cannot be reasonably acquired, the default values of Exhibit 10-14 may be used to provide an approximate estimate of construction zone capacity.

Lane-Width Consideration

The impact of lane width on general freeway operations is incorporated into the methodology of Chapter 11, Basic Freeway Segments, for determining free-flow speed. As free-flow speed affects capacity, it follows that restricted lane widths will negatively affect capacity.

As free-flow speeds are not estimated specifically for work or construction zones, it is appropriate to add an adjustment factor for the effect of lane widths narrower than 12 ft in a work or construction zone. The factor f_{LW} would be added to Equation 10-9, as shown in Equation 10-10:

Equation 10-10

$$c'_a = c_a \times f_{LW}$$

where c'_a is the adjusted capacity of the work or construction zone reflecting the impact of restricted lane width, in vehicles per hour, and all other variables are as previously defined.

The value of the adjustment factor f_{LW} is 1.00 for 12-ft lanes, 0.91 for lanes between 10.0 and 11.9 ft, and 0.86 for lanes between 9.0 and 9.9 ft. If lanes narrower than 9.0 ft are in use, local observations should be made to calibrate an appropriate adjustment.

Capacity Reductions due to Weather and Environmental Conditions

A number of studies have attempted to address the impacts of adverse weather and environmental conditions on the capacity of freeways. Comprehensive results for a range of conditions in Iowa, summarized in Exhibit 10-15, are provided elsewhere (*14*).

Exhibit 10-15
Capacity Reductions due to Weather and Environmental Conditions in Iowa

Type of Condition	Intensity of Condition	Percent Reduction in Capacity Average	Range
Rain	>0 ≤ 0.10 in./h	2.01	1.17–3.43
	>0.10 ≤ 0.25 in./h	7.24	5.67–10.10
	>0.25 in./h	14.13	10.72–17.67
Snow	>0 ≤ 0.05 in./h	4.29	3.44–5.51
	>0.05 ≤ 0.10 in./h	8.66	5.48–11.53
	>0.10 ≤ 0.50 in./h	11.04	7.45–13.35
	>0.50 in./h	22.43	19.53–27.82
Temperature	<50°F ≥ 34°F	1.07	1.06–1.08
	<34°F ≥ –4°F	1.50	1.48–1.52
	<–4°F	8.45	6.62–10.27
Wind	>10 ≤ 20 mi/h	1.07	0.73–1.41
	>20 mi/h	1.47	0.74–2.19
Visibility	<1 ≥ 0.50 mi	9.67	One site
	<0.50 ≤ 0.25 mi	11.67	One site
	<0.25 mi	10.49	One site

Source: Adapted from Agarwal et al. (*14*).

Additional information is available in the literature. Additional data and information on the impacts of rain on freeway capacity are provided elsewhere (*15, 16*), as are information on the effects of snow (*16*) and insights and information on the effects of fog (*17, 18*).

A study of capacity on German autobahns provides data on the difference between daytime and nighttime conditions on wet or dry pavements (*19*). Exhibit 10-16 summarizes these results.

Exhibit 10-16
Capacities on German Autobahns Under Various Conditions (veh/h/ln)

Freeway Lanes	Weekday or Weekend	Daylight Dry	Dark Dry	Daylight Wet	Dark Wet
6	Weekday (% change*)	1,489	1,299 (13%)	1,310 (12%)	923 (38%)
6	Weekend (% change*)	1,380	1,084 (21%)	1,014 (27%)	— —
4	Weekday (% change*)	1,739	1,415 (19%)	1,421 (18%)	913 (47%)
4	Weekend (% change*)	1,551	1,158 (25%)	1,104 (29%)	— —

Note: *Percent change from daylight, dry conditions for the same day of week.
Source: Adapted from Brilon and Ponzlet (*19*).

This exhibit is interesting in that the daylight, dry capacities of German autobahns are somewhat less than might be expected on U.S. freeways. This situation could be due to the higher speeds that prevail on the autobahns and heavy-vehicle presence, which are not reflected in these veh/h/ln statistics.

The daylight wet versus dry capacity reductions are greater in Exhibit 10-16 than those shown in Exhibit 10-15, which may again be a reflection of different driver behavior characteristics in Germany and the United States. Darkness alone has a significant impact on autobahn capacities. Since winter peak hours occur

when it is dark in many areas of the country, such reductions are important to recognize.

The difference between weekday and weekend capacities is also interesting and is on the order of 7% to 10% in Exhibit 10-16. This impact is generally reflected in the use of a driver-population factor f_p (see Chapter 11). Weekend driving populations may not be as familiar with the facility as weekday commuters. Even familiar users may not drive as aggressively on weekend recreational or other trips when the pressure of a specific schedule may be less than is present during the week.

Capacity Reductions due to Traffic Accidents or Vehicular Breakdowns

Capacity reductions due to traffic accidents or other incidents are generally short-lived, ranging from less than 1 h before they can be cleared to as long as 12 h for an accident involving severe injuries, fatalities, hazardous materials cleanup, or cleanup of other materials from vehicles involved in accidents.

One study (*20*) reported the mean duration of a traffic incident to be 37 min, with more than half the incidents lasting 30 min or less and 82% lasting less than 1 h.

Exhibit 10-17 summarizes the results of two studies (*21*, *22*) on the capacity impacts of lane blockages due to incidents, including accidents. An incident's effect on capacity depends on the proportion of the traveled roadway that is blocked and on the number of lanes on the freeway at that point.

Exhibit 10-17
Proportion of Freeway Segment Capacity Available Under Incident Conditions

Number of Lanes (One Direction)	Shoulder Disablement	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
2	0.95	0.81	0.35	0.00	N/A
3	0.99	0.83	0.49	0.17	0.00
4	0.99	0.85	0.58	0.25	0.13
5	0.99	0.87	0.65	0.40	0.20
6	0.99	0.89	0.71	0.50	0.26
7	0.99	0.91	0.75	0.57	0.36
8	0.99	0.93	0.78	0.63	0.41

In a blocked lane, the loss of capacity is likely to be greater than the proportion of the roadway that is blocked. A one-lane blockage on a two-lane directional freeway segment (50% of the roadway blocked) reduces capacity to 35% of the original value, for example. The added loss of capacity arises because drivers slow to look at the incident while they are abreast of it and are slow to react to the possibility of speeding up to move through the incident area.

The "rubbernecking" factor is also responsible for a reduction in capacity in the direction of travel opposite to that in which the accident or incident occurred. While no quantitative studies of this impact have been conducted, experience suggests that the severity of the accident or incident plays a significant role in the impact of rubbernecking. The reduction in capacity may range from 5% for a single-vehicle accident with one emergency vehicle present to as high as 25% for a multivehicle accident with several emergency vehicles.

Applying Capacity Reductions

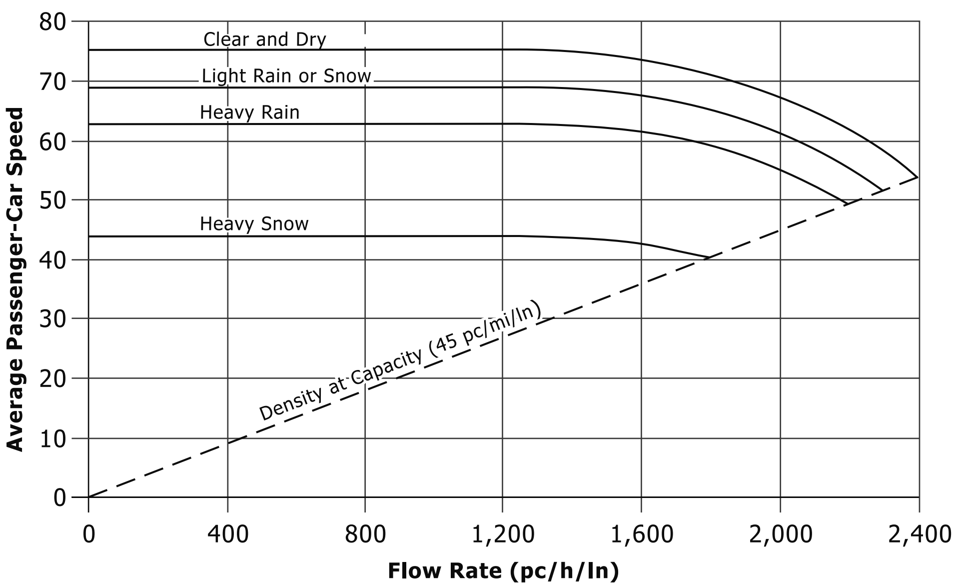
There are several ways to use the information on reduced capacities discussed in this section.

Quick approximations simply require that the capacity of each freeway facility segment (as estimated by using the methodologies of Chapters 11, 12, and 13) be reduced by all the impacts of work zones, weather, environment, and accidents or incidents that are present, in accord with the information provided here. The methodology continues using these reduced capacities.

If speed information is available, then the free-flow speed through the restricted capacity area can be used to select an appropriate speed–flow curve for analysis (from Chapter 11). The reduced free-flow speed results in a reduced capacity. An example of this approach is illustrated in Exhibit 10-18, which is based on speed data presented elsewhere (*16, 19*).

Exhibit 10-18
Illustration of Speed–Flow Curves for Different Weather Conditions

LIVE GRAPH
Click here to view

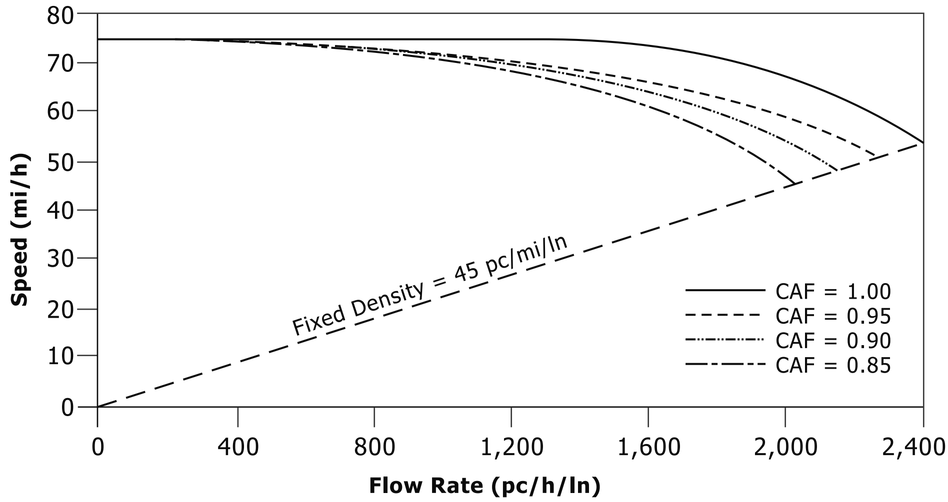


Note: Free-flow speed = 75 mi/h (base conditions).

For most temporary capacity reductions, the only information available relates to capacity. In most of these cases, speed conditions can be reasonably estimated. For example, in construction zones, a reduced speed limit is usually posted, and lower speeds can be expected to occur, particularly when actual construction operations are taking place. Likewise, for incidents, traffic naturally slows as drivers pass the incident site, where rubbernecking takes place. Exhibit 10-19 shows an example of modeling such cases on the basis of a downward-shifted speed–flow curve.

Exhibit 10-19
Illustration of Adjusted Speed–Flow Curves for Indicated Capacity Reductions

LIVE GRAPH
Click here to view



Note: Free-flow speed = 75 mi/h (base conditions); CAF = capacity adjustment factor (proportion of available capacity).

If the analyst has no interest in speeds, the capacity reduction could be modeled by using a fractional number of lanes that would reflect the new capacity of the roadway rather than the actual number of lanes present. For example, in the case of a four-lane directional freeway segment with two lanes blocked, Exhibit 10-17 indicates that only 25% of capacity would be available. This segment could be modeled as if only one lane were available through the incident (even though two are actually in use).

Some of the performance measures that result from this methodology, however, rely on speed. A simple approach that does not deal with speed consequences would result in an incomplete analysis. Consequently, an approach that uses modified speed–flow curves, as illustrated here, is recommended.

Step 5: Compute Demand-to-Capacity Ratios

Each cell of the time–space domain now contains an estimate of demand and capacity. A demand-to-capacity ratio can be calculated for each cell. The cell values must be carefully reviewed to determine whether all boundary cells have v_d/c ratios of 1.00 or less and to determine whether any cells in the interior of the time–space domain have v_d/c values greater than 1.00.

If any boundary cells have a v_d/c ratio greater than 1.00, further analysis may be significantly flawed:

1. If any cell in the first time interval has a v_d/c ratio greater than 1.00, there may have been oversaturated conditions in earlier time intervals without transfer of unsatisfied demand into the time–space domain of the analysis.
2. If any cell in the last time interval has a v_d/c ratio greater than 1.00, the analysis will be incomplete because the unsatisfied demand in the last time interval cannot be transferred to later time intervals.
3. If any cell in the last downstream segment has a v_d/c ratio greater than 1.00, there may be downstream bottlenecks that should be checked before

proceeding with the analysis. If any cell in the first segment has a v_d/c ratio greater than 1.00, then oversaturation will extend upstream of the defined freeway facility, but its effects will not be analyzed within the time–space domain.

These checks do not guarantee that the boundary cells will not show v_d/c ratios greater than 1.00 later in the analysis. If these initial checks reveal boundary cells with v_d/c ratios greater than 1.00, then the time–space domain of the analysis should be adjusted to eliminate the problem.

As the analysis of the time–space domain proceeds, subsequent demand shifts may cause some boundary cell v_d/c ratios to exceed 1.00. In these cases, the problem should be reformulated or alternative tools applied. Most alternative tools will have the same problem if the boundary conditions experience congestion.

Another important check is to observe whether any cell in the interior of the time–space domain has a v_d/c ratio greater than 1.00. There are two possible outcomes:

1. If all cells have v_d/c ratios of 1.00 or less, then the entire time–space domain contains undersaturated flow, and the analysis is greatly simplified.
2. If any cell in the time–space domain has a v_d/c ratio greater than 1.00, then the time–space domain will contain both undersaturated and oversaturated cells. Analysis of oversaturated conditions is much more complex because of the interactions between freeway segments and the shifting of demand in both time and space.

If Case 1 exists, the analysis moves to Step 6A. If Case 2 exists, the analysis moves to Step 6B.

The v_d/c ratio for all on-ramps and off-ramps should also be examined. If an on-ramp demand exceeds the on-ramp capacity, the ramp demand flow rates should be adjusted to reflect capacity. Off-ramps generally fail because of deficiencies at the ramp–street junction. They may be analyzed by procedures in Chapters 18–22, depending on the type of traffic control used at the ramp–street junction. These checks are done manually, and inputs to this methodology must be revised accordingly.

Steps 6A and 6B: Compute Undersaturated (6A) / Oversaturated (6B) Service Measures and Other Performance Measures

The analysis begins in the first cell in the upper-left corner of the time–space domain (the first segment in the first time interval) and continues downstream along the freeway facility for each segment in the first time interval. The analysis then returns to the first upstream segment in the second time interval and continues downstream along the freeway for each segment in the second time interval. This process continues until all cells in the time–space domain have been analyzed.

As each cell is analyzed in turn, its v_d/c ratio is checked. If the v_d/c ratio is 1.00 or less, the cell is not a bottleneck and is able to handle all traffic demand that

wishes to enter. The process is continued in the order noted in the previous paragraph until a cell with a v_d/c ratio greater than 1.00 is encountered. Such a cell is labeled as a bottleneck. Because it cannot handle a flow greater than its capacity, the following impacts will occur:

1. The v_d/c ratio of the bottleneck cell will be exactly 1.00, as the cell processes a flow rate equal to its capacity.
2. Flow rates for all cells downstream of the bottleneck must be adjusted downward to reflect the fact that not all the demand flow at the bottleneck gets through. Downstream cells are subject to demand starvation due to the bottleneck.
3. The unsatisfied demand at the bottleneck cell must be stored in the upstream segments. Flow conditions and performance measures in these upstream cells are affected. Shock wave analysis is applied to estimate these impacts.
4. The unsatisfied demand stored upstream of the bottleneck cell must be transferred to the next time interval. This transfer is accomplished by adding the unsatisfied demand by desired destination to the origin–destination table of the next time interval.

This four-step process is implemented for each bottleneck encountered, following the specified sequence of cell analysis. If no bottlenecks are identified, the entire domain is undersaturated, and the sequence of steps for oversaturated conditions is not applied.

If a bottleneck is severe, the storage of unsatisfied demand may extend beyond the upstream boundary of the freeway facility or beyond the last time interval of the time–space domain. In such cases, the analysis will be flawed, and the time–space domain should be reconstituted.

After all demand shifts (in the case of one or more oversaturated cells) are estimated, each cell is analyzed by the methodologies of Chapter 11, Basic Freeway Segments; Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments. Facility service and performance measures may then be estimated.

Undersaturated Conditions

For undersaturated conditions, the process is straightforward. Because there are no cells with v_d/c ratios greater than 1.00, the flow rate in each cell, v_a, is equal to the demand flow rate, v_d. Each segment analysis using the methodologies of Chapters 11, 12, and 13 will result in estimating a density D and a space mean speed S.

When the analysis moves from isolated segments to a system, additional constraints may be necessary. A maximum-achievable-speed constraint is imposed to limit the prediction of speeds in segments downstream of a segment experiencing low speeds. This constraint prevents large speed fluctuations from segment to segment when the segment methodologies are directly applied. This process results in some changes in the speeds and densities predicted by the segment methodologies.

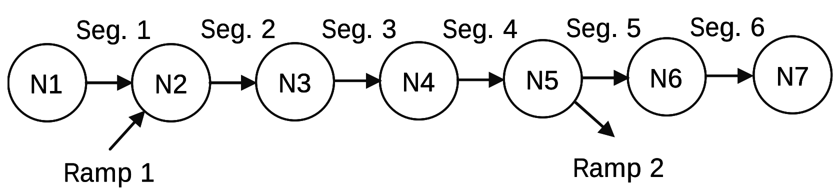
For each time interval, Equation 10-2 is used to estimate the average density for the defined freeway facility. This result is compared with the criteria of Exhibit 10-7 to determine the facility LOS for the time period. Each time period will have a separate LOS. Although LOS is not averaged over time intervals, if desired, density can be averaged over time intervals.

Oversaturated Conditions

Once oversaturation is encountered, the methodology changes its temporal and spatial units of analysis. The spatial units become nodes and segments, and the temporal unit moves from a time interval of 15 min to smaller time periods, as recommended in Chapter 25, Freeway Facilities: Supplemental.

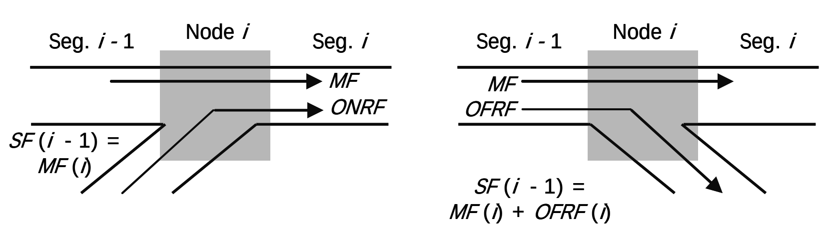
Exhibit 10-20 illustrates the node–segment concept. A node is defined as the junction of two segments. Given that there is a node at the beginning and end of the freeway facility, there will always be one more node than the number of segments on the facility.

Exhibit 10-20
Node–Segment Representation of a Freeway Facility



The numbering of nodes and segments begins at the upstream end of the defined freeway facility and moves to the downstream end. The segment upstream of node i is numbered $i - 1$, and the downstream segment is numbered i, as shown in Exhibit 10-21.

Exhibit 10-21
Mainline and Segment Flow at On- and Off-Ramps



Note: *SF* = segment flow, *MF* = mainline flow, *ONRF* = on-ramp flow, and *OFRF* = off-ramp flow.

The oversaturated analysis moves from the first node to each downstream node for a time step. After the analysis for the first time step is complete, the same nodal analysis is performed for each subsequent time step.

When oversaturated conditions exist, many flow variables must be adjusted to reflect the upstream and downstream effects of bottlenecks. These adjustments are explained in general terms in the sections that follow and are fully detailed in Chapter 25.

Flow Fundamentals

As noted previously, segment flow rates must be calculated for each time step. They are used to estimate the number of vehicles on each segment at the

end of every time step. The number of vehicles on each segment is used to track queue accumulation and discharge and to estimate the average segment density.

The conversion from standard 15-min time intervals to time steps (of lesser duration) occurs during the first oversaturated interval. Time steps are then used until the analysis is complete. This transition to time steps is critical because, at certain points in the methodology, future performance is estimated from past performance of an individual variable. Use of time steps also allows for a more accurate estimation of queues.

Service and other performance measures for oversaturated conditions use a simplified, linear flow–density relationship, as detailed in Chapter 25.

Segment Initialization

To estimate the number of vehicles on each segment for each time step under oversaturated conditions, it is necessary to begin the process with the appropriate number of vehicles in each segment. Determining this number is referred to as *segment initialization.*

A simplified queuing analysis is initially performed to account for the effects of upstream bottlenecks. The bottlenecks limit the number of vehicles that can proceed downstream.

To obtain the proper number of vehicles on each segment, the expected demand is calculated from the demands for and capacities of the segment, including the effects of all upstream segments. The expected demand represents the flow that would arrive at each segment if all queues were stacked vertically (i.e., as if the queues had no upstream impacts). For all segments upstream of a bottleneck, the expected demand will equal the actual demand.

For the bottleneck segment and all further downstream segments, a capacity restraint is applied at the bottleneck when expected demand is computed. From the expected segment demand, the background density can be obtained for each segment by using the appropriate estimation algorithms from Chapters 11, 12, and 13.

Mainline Flow Calculation

Flows analyzed in oversaturated conditions are calculated for every time step and are expressed in vehicles per time step. They are analyzed separately on the basis of the origin and destination of the flow across the node. The following flows are defined:

1. The flow from the mainline upstream segment $i-1$ to the mainline downstream segment i is the mainline flow MF.
2. The flow from the mainline to an off-ramp is the off-ramp flow $OFRF$.
3. The flow from an on-ramp to the mainline is the on-ramp flow $ONRF$.

Each of these flows is illustrated in Exhibit 10-21.

Mainline Input

The mainline input is the number of vehicles that wish to travel through a node during the time step. The calculation includes the effects of bottlenecks

upstream of the subject node. The effects include the metering of traffic during queue accumulation and the presence of additional vehicles during queue discharge.

The mainline input is calculated by taking the number of vehicles entering the node upstream of the analysis node, adding on-ramp flows or subtracting off-ramp flows, and adding the number of unserved vehicles on the upstream segment. The result is the maximum number of vehicles that desire to enter a node during a time step.

Mainline Output

The mainline output is the maximum number of vehicles that can exit a node, constrained by downstream bottlenecks or by merging traffic. Different constraints on the output of a node result in three different types of mainline outputs (MO1, MO2, and MO3).

- *Mainline output from ramps (MO1):* MO1 is the constraint caused by the flow of vehicles from an on-ramp. The capacity of an on-ramp flow is shared by two competing flows: flow from the on-ramp and flow from the mainline. The total flow that can pass the node is estimated as the minimum of the segment i capacity and the mainline outputs (MO2 and MO3) calculated in the preceding time step.
- *Mainline output from segment storage (MO2):* The output of mainline flow through a node is also constrained by the growth of queues on the downstream segment. The presence of a queue limits the flow into the segment once the queue reaches its upstream end. The queue position is calculated by shock wave analysis. The MO2 limitation is determined first by calculating the maximum number of vehicles allowed on a segment at a given queue density. The maximum flow that can enter a queued segment is the number of vehicles leaving the segment plus the difference between the maximum number of vehicles allowed on a segment and the number of vehicles already on the segment. The queue density is determined from the linear congested portion of the density–flow relationship shown in Chapter 25.
- *Mainline output from front-clearing queue (MO3):* The final limitation on exiting mainline flows at a node is caused by front-clearing downstream queues. These queues typically occur when temporary incidents clear. Two conditions must be satisfied: (*a*) the segment capacity (minus the on-ramp demand if present) for the current time interval must be greater than the segment capacity (minus on-ramp demand) in the preceding time interval, and (*b*) the segment capacity minus the ramp demand for the current time interval must be greater than the segment demand in the same time interval. Front-clearing queues do not affect the segment throughput (which is limited by queue throughput) until the recovery wave has reached the upstream end of the segment. The shock wave speed is estimated from the slope of the line connecting the bottleneck throughput and the segment capacity points.

Mainline Flow

The mainline flow across node i is the minimum of the following variables:

- Node i mainline input,
- Node i MO2,
- Node i MO3,
- Segment $i - 1$ capacity, and
- Segment i capacity.

Determining On-Ramp Flow

The on-ramp flow is the minimum of the on-ramp input and output. Ramp input in a time step is the ramp demand plus any unserved ramp vehicles from a previous time step.

On-ramp output is limited by the ramp roadway capacity and the ramp-metering rate. It is also affected by the volumes on the mainline segments. The latter is a very complex process that depends on the various flow combinations on the segment, the segment capacity, and the ramp roadway volumes. Details of the calculations are presented in Chapter 25.

Determining Off-Ramp Flow

The off-ramp flow is determined by calculating a diverge percentage based on the segment and off-ramp demands. The diverge percentage varies only by time interval and remains constant for vehicles that are associated with a particular time interval. If there is an upstream queue, then traffic to this off-ramp may be metered. This will cause a decrease in the off-ramp flow. When vehicles that were metered arrive in the next time interval, they use the diverge percentage associated with the preceding time interval. This methodology ensures that all off-ramp vehicles prevented from exiting during the presence of a bottleneck are appropriately discharged in later time intervals.

Determining Segment Flow

Segment flow is the number of vehicles that flow out of a segment during the current time step. These vehicles enter the current segment either to the mainline or to an off-ramp at the current node as shown in Exhibit 10-20. The number of vehicles on each segment in the current time step is calculated with the following information:

- The number of vehicles that were in the segment in the previous time step,
- The number of vehicles that entered the segment in the current time step, and
- The number of vehicles that can leave the segment in the current time step.

Because the number of vehicles that leave a segment must be known, the number of vehicles on the current segment cannot be determined until the upstream segment is analyzed.

The number of unserved vehicles stored on a segment is calculated as the difference between the number of vehicles on the segment and the number of vehicles that would be on the segment at the background density.

Determining Segment Service Measures

In the last time step of a time interval, the segment flows in each time step are averaged over the time interval, and the service measures for each segment are calculated. If there were no queues on a particular segment during the entire time interval, then the performance measures are calculated from Chapters 11, 12, and 13 as appropriate.

If there was a queue on the current segment during the time interval, then the performance measures are calculated in four steps:

1. The average number of vehicles over a time interval is calculated for each segment.
2. The average segment density is calculated by taking the average number of vehicles in all time steps (in the time interval) and dividing it by the segment length.
3. The average speed on the current segment during the current time interval is calculated as the ratio of segment flow to density.
4. The final segment performance measure is the length of the queue at the end of the time interval (if one exists), which is calculated by using shock wave theory.

On-ramp queue lengths can also be calculated. A queue will form on the on-ramp roadway only if the flow is limited by a meter or by freeway traffic in the gore area. If the flow is limited by the ramp roadway capacity, unserved vehicles will be stored on a facility upstream of the ramp roadway, most likely a surface street. The methodology does not account for this delay. If the queue is on a ramp roadway, its length is calculated by using the difference in background and queue densities.

Step 7: Compute Freeway Facility Service Measures and Other Performance Measures by Time Interval

The previously discussed traffic performance measures can be aggregated over the length of the defined freeway facility for each time interval. Aggregations over the entire time–space domain of the analysis are also mathematically possible, although LOS is defined only for 15-min time intervals.

Freeway facility LOS is defined for each time interval included in the analysis. An average density for each time interval, weighted by length of segments and numbers of lanes in segments, is calculated (with Equation 10-2) and used to compare with the criteria of Exhibit 10-7.

3. APPLICATIONS

Specific computational steps for the freeway facility methodology were conceptually discussed and presented in this chapter's methodology section. Additional computational details are provided in Chapter 25, Freeway Facilities: Supplemental.

This chapter's methodology is sufficiently complex to require software for its application. Even for fully undersaturated analyses, the number and complexity of computations make it difficult and extremely time-consuming to analyze a case manually. Oversaturated analyses are considerably more complex, and manual solutions would be impractical. The computational engine for this methodology is FREEVAL-2010. A complete user's guide and executable spreadsheet are available in the Technical Reference Library in Volume 4.

OPERATIONAL ANALYSIS

The only mode in which the methodology can be directly implemented is operational analysis—that is, given a complete description of a freeway facility, its component segment geometries, and all relevant demand flow rates, a complex analysis is conducted of each segment, and of the freeway facility, by time interval. Outputs will include segment flow rates, densities, and average speeds as well as average facility density and speed for each time interval. By using the estimated facility density for each time interval, a facility LOS can be assigned.

Exhibit 10-22 shows the data inputs that are required for an operational analysis of a freeway facility.

Exhibit 10-22
Required Input Data for Freeway Facility Analysis

Geometric Data for Each Section
• Section length (ft) • Mainline number of lanes • Mainline average lane width (ft) • Mainline lateral clearance (ft) • Terrain (level, rolling, or mountainous), or specific grade (% grade, length in mi) • Ramp number of lanes • Ramp acceleration or deceleration lane length (ft) • Existence of independent HOV lane
Traffic Characteristic Data for Each Segment
• Mainline free-flow speed (mi/h), optional • Vehicle occupancy (passengers/veh) • Percent trucks and buses (%) • Percent RVs (%) • Driver population (commuter or recreational) • Ramp free-flow speeds (mi/h)
Demand Data for Each Segment
• Mainline entry demand for each time interval (veh/h) • On-ramp demands for each time interval (veh/h) • Off-ramp demands for each time interval (veh/h) • Weaving demand on weaving segments, by movement (veh/h) • HOV lane demand (veh/h), if present

Where all data are not readily available or collectable, the analysis may be supplemented by using consistent default values for each segment. Lists and discussions of default values are found in Chapter 11, Basic Freeway Segments;

Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments.

Performance measures output by the methodology for individual segments and the facility (for a given time interval) include the following:

- Average speed (mi/h),
- Average density (pc/mi/ln),
- Vehicle miles of travel,
- Vehicle hours of travel, and
- Travel time (min/veh).

Chapter 25 details facilitywide performance measure calculations by time interval.

PLANNING, PRELIMINARY ENGINEERING, AND DESIGN ANALYSIS

This methodology cannot be directly used in planning, preliminary engineering, and design applications. However, for generalized planning, Exhibit 10-8 (urban freeways) and Exhibit 10-9 (rural freeways) provide daily-service-volume tables for a variety of typical freeway conditions. These tables may be applied for general evaluations of a number of freeway facilities in a specified region. They should not be used for directly evaluating a specific freeway facility or for developing detailed facility improvement plans. A full operational analysis would normally be applied to any freeway facility identified as potentially needing improvement.

Preliminary engineering and design applications of the methodology are possible by using the segment procedures described in Chapters 11, 12, and 13. Various geometric scenarios can be evaluated and compared by using a travel demand matrix and the facility methodology on the basis of the segment results.

TRAFFIC MANAGEMENT STRATEGIES

The freeway facilities methodology has incorporated procedures for assessing a variety of traffic management strategies. The methodology permits modifying previously calculated cell demands or capacities (or both) within the time–space domain to assess a traffic management strategy or a combination of strategies.

1. A growth factor parameter has been incorporated to evaluate traffic performance when traffic demands are higher or lower than the demand calculated from the traffic counts. This parameter would be used to undertake a sensitivity analysis of the effect of demand on freeway performance and to evaluate future scenarios. In these cases, all cell demand estimates are multiplied by the growth factor parameter.
2. The effect of a predetermined ramp-metering plan can be evaluated by modifying the ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to the desired metering rate. This feature permits evaluating a predetermined ramp-metering plan and experimenting to obtain an improved ramp-metering plan.

3. Freeway design improvements can be evaluated with this methodology by modifying the design features of any portion of the freeway facility. For example, the effect of adding an auxiliary lane at a critical location or adding merging or diverging lanes can be assessed.
4. Reduced-capacity situations can be investigated. The capacity in any cell or cells of the time–space domain can be reduced to represent situations such as construction and maintenance activities, adverse weather, and traffic accidents and vehicle breakdowns.
5. User demand responses such as spatial, temporal, modal, and total demand responses caused by a traffic management strategy are not automatically incorporated into the methodology. On viewing the new freeway traffic performance results, the user can modify the demand input manually to evaluate the effect of anticipated demand responses.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for applying alternative tools to the analysis of freeway facilities. Additional information on this topic may be found in Chapter 25, Freeway Facilities: Supplemental.

Strengths of the HCM Procedure

This chapter's procedures were based on extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent a consensus of experts. Specific strengths of the HCM freeway facilities procedures include the following:

- They provide more detailed algorithms for considering geometric elements of the facility (such as lane and shoulder width).
- They provide capacity estimates for each segment of the facility, which simulation tools do not provide directly (and in some cases may require as an input).
- The capacity can be explicitly adjusted to account for weather conditions, lighting conditions, work zone setup and activity, and incidents.
- The calculation of key performance measures, such as speed and density, is transparent. Simulation tools often use statistics accumulated over the simulation period to derive various link or time-period-specific results, and the derivation of these results may not be obvious. Thus, the user of a simulation tool must know exactly which measure is being reported (e.g., space mean speed versus time mean speed). Furthermore, simulation tools may apply these measures in ways different from the HCM to arrive at other measures.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

Freeway facilities can be analyzed with a variety of stochastic and deterministic simulation tools. These tools can be useful in analyzing the extent of congestion when there are failures within the simulated facility range and when interaction with other freeway segments and other facilities is present.

Exhibit 10-23 provides a list of the limitations stated earlier in this chapter, along with their potential for improved treatment by alternative tools.

Exhibit 10-23
Limitations of the HCM Freeway Facilities Analysis Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Changes in travel time caused by vehicles using alternate routes	Modeled explicitly by dynamic traffic assignment tools
Multiple overlapping bottlenecks	Modeled explicitly by simulation tools
User-demand responses (spatial, temporal, modal)	Modeled explicitly by dynamic traffic assignment tools
Systemwide oversaturated flow conditions	Modeled explicitly by simulation tools
First/last time interval or first/last segment demand-to-capacity ratio > 1.0	Modeled explicitly by simulation tools, except that a simulation analysis may also be inaccurate if it does not fully account for a downstream bottleneck that causes congestion in the last segment during the last time period
Interaction between managed lanes and mixed-flow lanes	Modeled explicitly by some simulation tools

Additional Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating a variety of performance measures for individual segments along a freeway facility, and the entire facility, given each segment's traffic demand and characteristics. The following performance measures are reported by the freeway facilities procedure:

- Travel time,
- Free-flow travel time,
- Traffic delay,
- Vehicle miles of travel,
- Person miles of travel,
- Speed, and
- Density (segment only).

Alternative tools can offer additional performance measures, such as queue lengths, fuel consumption, vehicle emissions, and operating costs. As with most other procedural chapters in the HCM, simulation outputs—especially graphics-based presentations—may provide details on point problems that might go unnoticed with a macroscopic analysis.

Development of HCM-Compatible Performance Measures Using Alternative Tools

LOS for all types of freeway segments is estimated by the density of traffic (pc/mi/ln) on each segment. The guidance provided in Chapter 11, Basic Freeway

Segments, for developing compatible density estimates applies to freeway facilities as well.

With the exception of free-flow travel time, the additional performance measures listed above that are produced by the procedures in this chapter are also produced by typical simulation tools. For the most part, the definitions are compatible, and, subject to the precautions and calibration requirements that follow, the performance measures from alternative tools may be considered equivalent to those that are produced by the procedures in this chapter.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

To better determine when simulation of a freeway facility may be more appropriate than an HCM analysis, the fundamental differences between the two approaches must be understood. The HCM and simulation analysis approaches are reviewed in the following subsections.

HCM Approach

The HCM analysis procedure uses one of two approaches—one for undersaturated conditions and one for oversaturated conditions. For undersaturated conditions—that is, v_d/c is less than 1.0 for all segments and time periods—the approach is generally disaggregate. In other words, the facility is subdivided into segments corresponding to basic freeway, weaving, and merge/diverge segments, and the LOS results are reported for individual segments on the basis of the analysis procedures of Chapters 11, 12, and 13, respectively. However, LOS results are not reported for the facility as a whole.

For oversaturated conditions, the facility is analyzed in a different manner. First, the facility is considered in its entirety rather than at the individual segment level. Second, the analysis time interval, typically 15 min, is subdivided into time steps of 15 to 60 s, depending on the length of the shortest segment. This approach is necessary so that flows can be reduced to capacity levels at bottleneck locations and queues can be tracked in space and time. For oversaturated segments, the average segment density is calculated by dividing the average number of vehicles for all time steps (in the time interval) by the segment length. The average segment speed is calculated by dividing the average segment flow rate by the average segment density. Facilitywide performance measures are calculated by aggregating segment performance measures across space and time, as outlined in Chapter 25. A LOS for the facility is assigned on the basis of density for each time interval.

When the oversaturation analysis procedure is applied, if any segment is undersaturated for an entire time interval, its performance measures are calculated according to the appropriate procedure in Chapters 11, 12, and 13.

Simulation Approach

Simulation tools model the facility in its entirety and from that perspective have some similarity to the oversaturated analysis approach of the HCM. Microscopic simulation tools operate similarly under both saturated and undersaturated conditions, tracking each vehicle through time and space and

generally handling the accumulation and queuing of vehicles in saturated conditions in a realistic manner. Macroscopic simulation tools vary in their treatment of saturated conditions. Some tools do not handle oversaturated conditions at all, while others may queue vehicles in the vertical, rather than horizontal, dimension. These tools may still provide reasonably accurate results under slightly oversaturated conditions, but the results will clearly be invalid for heavily congested conditions.

The treatment of oversaturated conditions is a fundamental issue that must be understood when considering whether to apply simulation in lieu of the HCM for analysis of congested conditions. A review of simulation modeling approaches is beyond the scope of this document. More detailed information on the topic may be found in the Technical Reference Library in Volume 4.

Adjustment of Simulation Parameters to the HCM Results

Some calibration is generally required before an alternative tool can be used effectively to supplement or replace the HCM procedure. The following subsections discuss key variables that should be checked for consistency with the HCM procedure values.

Capacity

In the HCM, capacity is a function of the specified free-flow speed (which can be adjusted by lane width, shoulder width, and ramp density). In a simulation tool, capacity is typically a function of the specified minimum vehicle entry headway (into the system) and car-following parameters (assuming microscopic simulation).

While the determination of capacity for a basic freeway segment is clearly described in Chapter 11, this chapter does not offer specific guidance on determining the appropriate capacity for different segment types within a facility, other than to refer the reader to the individual chapters (basic segments, weaving segments, merge segments, diverge segments) for appropriate capacity values. The HCM specifies the capacity of a freeway facility in units of veh/h rather than pc/h.

In macroscopic simulation tools, capacity is generally an input. Thus, for this situation, it is straightforward to match the simulation capacity to the HCM capacity. Microscopic simulation tools, however, do not have an explicit capacity input. Most microscopic tools provide an input that affects the minimum separation for the generation of vehicles into the system. Therefore, specifying a value of 1.5 s for this input will result in a maximum vehicle entry rate of 2,400 (3,600/1.5) veh/h/ln. Once vehicles enter the system, vehicle headways are governed by the car-following model. Thus, given other factors and car-following model constraints, the maximum throughput on any one segment may not reach this value. Consequently, some experimenting is usually necessary to find the right minimum entry separation value to achieve a capacity value comparable with that in the HCM. Again, the analyst needs to be careful of the units being used for capacity in making comparisons.

The other issue to be aware of is that, while geometric factors such as lane and shoulder width affect the free-flow speed (which in turn affects capacity) in the HCM procedure, some simulation tools do not account for these effects, or they may account for other factors, such as horizontal curvature, that the HCM procedure does not consider.

Lane Distribution

In the HCM procedure, there is an implicit assumption that, for any given vehicle demand, the vehicles are evenly distributed across all lanes of a basic freeway segment. For merge and diverge segments, the HCM procedure includes calculations to determine how vehicles are distributed across lanes as a result of merging or diverging movements. For weaving segments, there is not an explicit determination of flow rates in particular lanes, but consideration of weaving and nonweaving flows and the number of lanes available for each is an essential element of the analysis procedure.

In simulation tools, the distribution of vehicles across lanes is typically specified only for the entry point of the network. Once vehicles have entered the network, they are distributed across lanes according to car-following and lane-changing logic. This input value should reflect field data if they are available. If field data indicate an imbalance of flows across lanes, this situation may lead to a difference between the HCM and simulation results. If field data are not available, specifying an even distribution of traffic across all lanes is probably reasonable for networks that begin with a long basic segment. If there is a ramp junction within a short distance downstream of the entry point of the network, setting the lane distribution values to be consistent with those from Chapter 13 of the HCM will likely yield more consistent results.

Traffic Stream Composition

The HCM deals with the presence of non–passenger car vehicles in the traffic stream by applying passenger car equivalent values. These values are based on the percentage of trucks, buses, and RVs in the traffic stream as well as type of terrain (grade profile and its length). Thus, the traffic stream is converted into some equivalent number of passenger cars only, and the analysis results are based on flow rates in these units.

In the case of stochastic-based simulators, the generated vehicle type percentages may only approximate the specified percentages.

Simulation tools deal with the traffic stream composition just as it is specified; that is, the specific percentages of each vehicle type are generated into and moved through the system according to their specific vehicle attributes (e.g., acceleration and deceleration capabilities). Thus, simulation, particularly microscopic simulation, results likely better reflect the effects of non–passenger car vehicles on the traffic stream. Although in some instances the passenger car equivalent values contained in the HCM were developed from simulation data, simplifying assumptions made to make them implementable in an analytical procedure result in some loss of fidelity in the treatment of different vehicle types.

Furthermore, it should be recognized that the HCM procedures do not explicitly account for differences in driver types. Microscopic simulation tools explicitly provide for a range of driver types and allow a number of factors

related to driver type to be modified (e.g., free-flow speed, gap acceptance threshold). However, it should also be recognized that the empirical data some HCM procedures are based on include the effects of the various driver types present in traffic streams.

Free-Flow Speed

In the HCM, free-flow speed is either measured in the field or estimated with calibrated predictive algorithms. In simulation, free-flow speed is almost always an input value. Where field measurements are not available, simulation users may wish to use the HCM predictive algorithms to estimate free-flow speed.

Step-by-Step Recommendations for Applying Alternative Tools

General guidance for applying alternative tools is provided in Chapter 6, HCM and Alternative Analysis Tools. The chapters that cover specific types of freeway segments offer more detailed step-by-step guidance specific to those segments. All the segment-specific guidance applies to freeway facilities, which are configured as combinations of different segments.

The first step is to determine whether the facility can be analyzed satisfactorily by the procedures described in this chapter. If the facility contains geometric or operational elements beyond the scope of these procedures, then an alternative tool should be selected. The steps involved in the application will depend on the reason(s) for choosing an alternative tool. In some cases, the step-by-step segment guidance will cover the situation adequately. In more complex cases (e.g., those that involve integrated analysis of a freeway corridor), more comprehensive guidance from one or more documents in the Technical Reference Library in Volume 4 may be needed.

Sample Calculations Illustrating Alternative Tool Applications

The limitations of this chapter's procedures are mainly related to the lack of a comprehensive treatment of the interaction between segments and facilities. Many of these limitations can be addressed by simulation tools, which generally take a more integrated approach to the analysis of complex networks of freeways, ramps, and surface street facilities. Supplemental examples illustrating interactions between segments are presented in Chapter 26, Freeway and Highway Segments: Supplemental, and Chapter 34, Interchange Ramp Terminals: Supplemental. A comprehensive example of the application of simulation tools to a major freeway reconstruction project is presented as Case Study 6 in the HCM Applications Guide located in Volume 4.

4. EXAMPLE PROBLEMS

Exhibit 10-24
List of Example Problems

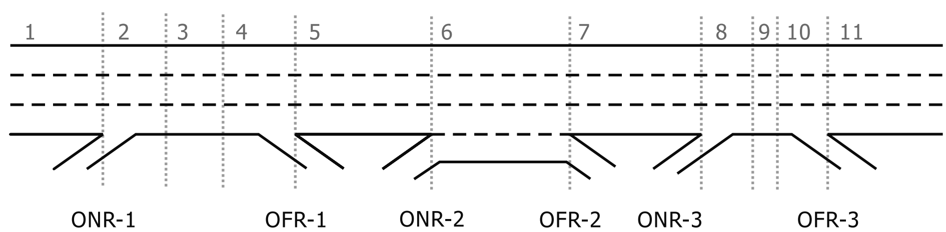
Example Problem	Description	Application
1	Evaluation of an undersaturated facility	Operational analysis
2	Evaluation of an oversaturated facility	Operational analysis
3	Capacity improvements to an oversaturated facility	Operational analysis

EXAMPLE PROBLEM 1: EVALUATION OF AN UNDERSATURATED FACILITY

The Facility

The subject of this operational analysis is an urban freeway facility 6 mi long and composed of 11 individual analysis segments, as shown in Exhibit 10-25.

Exhibit 10-25
Freeway Facility in Example Problem 1



The facility has three on-ramps and three off-ramps. Geometric details are given in Exhibit 10-26.

Exhibit 10-26
Geometry of Directional Freeway Facility for Example Problem 1

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	3	3	3	3	3

Note: B = basic freeway segment, W = weaving segment, ONR = on-ramp (merge) segment, OFR = off-ramp (diverge) segment, R = overlapping ramp segment.

The on- and off-ramps in Segment 6 are connected by an auxiliary lane and the segment may therefore operate as a weaving segment, depending on traffic patterns. The separation of the on-ramp in Segment 8 and the off-ramp in Segment 10 is less than 3,000 ft. Since the ramp influence area of on-ramps and off-ramps is 1,500 ft, according to Chapter 13, the segment affected by both ramps is analyzed as a separate overlapping ramp segment (Segment 9), labeled "R."

The analysis question at hand is the following: What is the operational performance and LOS of the directional freeway facility shown in Exhibit 10-25?

The Facts

In addition to the information contained in Exhibit 10-25 and Exhibit 10-26, the following characteristics of the freeway facility are known:

Heavy vehicles	=	5% trucks, 0% RVs (all movements);
Driver population	=	regular commuters;
FFS	=	60 mi/h (all mainline segments);
Ramp *FFS*	=	40 mi/h (all ramps);
Acceleration lane length	=	500 ft (all ramps);
Deceleration lane length	=	500 ft (all ramps);
D_{jam}	=	190 pc/mi/ln;
c_{IFL}	=	2,300 pc/h/ln (for *FFS* = 60 mi/h);
L_s	=	1,640 ft (for Weaving Segment 6);
TRD	=	1.0 ramp/mi;
Terrain	=	level; and
Analysis duration	=	75 min (divided into five 15-min intervals).

Comments

The facility was segmented into analysis segments on the basis of the guidance given in this chapter. The facility shown in Exhibit 10-25 initially depicts seven freeway *sections* (measured between ramps) that are divided into 11 analysis *segments.* The facility contains each of the possible segment types for illustrative purposes, including basic segment (B), weaving segment (W), merge segment (ONR), diverge segment (OFR), and overlapping ramp segment (R). The input data contain the required information needed for each of the segment methodologies.

The classification of the weave in Segment 6 is preliminary until it is determined whether the segment operates as a weave. For this purpose, the short length must be compared with the maximum length for weaving analysis to determine whether the Chapter 12, Weaving Segments, methodology or the Chapter 11, Basic Freeway Segments, methodology is applicable. The short length of the weaving segment used for calculation is shorter than the weaving influence area over which the calculated speed and density measures are applied.

Chapter 11 must be consulted to find appropriate values for the heavy vehicle adjustment factor f_{HV} and the driver population adjustment factor f_p. FREEVAL-2010 automatically determines these adjustment factors for general terrain conditions, but user input is needed for specific upgrades and composite grades.

All input parameters have been specified, so default values are not needed. Fifteen-minute demand flow rates are given in vehicles per hour under prevailing conditions. These demands must be converted to passenger cars per

hour under equivalent ideal conditions for use in the parts of the methodology related to segment LOS estimation.

Step 1: Input Data

Traffic demand inputs for all 11 segments and five analysis intervals are given in Exhibit 10-27.

Exhibit 10-27
Demand Inputs for Example Problem 1

Time Step (15 min)	Entering Flow Rate (veh/h)	Ramp Flow Rates by Time Period (veh/h)							Exiting Flow Rate (veh/h)
		ONR1	ONR2*		ONR3	OFR1	OFR2	OFR3	
1	4,505	450	540	(50)	450	270	360	270	5,045
2	4,955	540	720	(100)	540	360	360	270	5,765
3	5,225	630	810	(150)	630	270	360	450	6,215
4	4,685	360	360	(80)	450	270	360	270	4,955
5	3,785	180	270	(50)	270	270	180	180	3,875

* Numbers in parentheses indicate ONR-2 to OFR-2 demand flow rates in Weaving Segment 6.

The volumes in Exhibit 10-27 represent the 15-min demand flow rates on the facility as determined from field observations or other sources. The actual volume served in each segment will be determined by the methodology. The demand flows are given for the extended time–space domain, consistent with this chapter's recommendations. Peaking occurs in the third 15-min period. Since inputs are in the form of 15-min flow rates, no peak hour factor adjustment is necessary. Additional geometric and traffic-related inputs are as specified in Exhibit 10-25 and the facts section of the problem statement.

Step 2: Demand Adjustments

The traffic flows in Exhibit 10-27 are already given in the form of actual demands. Therefore, no additional demand adjustment is necessary, since the flows represent true demand. Demand adjustment is necessary only if field-measured volumes are used that may be affected by upstream congestion (bottleneck) on the facility. The methodology (and FREEVAL-2010) assume that the user inputs true demand flows.

Step 3: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments. The resulting capacities are shown in Exhibit 10-28. Since the capacity of a weaving segment depends on traffic patterns, including the weaving ratio, it varies by time period. The remaining segment capacities are constant in all five time intervals. The capacities for Segments 1–5 and 7–11 are the same, since the segments have the same basic cross section. The units shown are in vehicles per hour.

Exhibit 10-28
Segment Capacities for Example Problem 1

Time Step	Capacities (veh/h) by Segment 1	2	3	4	5	6	7	8	9	10	11
1						8,252					
2						8,261					
3	6,732	6,732	6,732	6,732	6,732	8,303	6,732	6,732	6,732	6,732	6,732
4						8,382					
5						8,442					

Step 4: Adjust Segment Capacities

This step typically allows the user to adjust capacities of specific segments or time periods to model the effects of short-term work zones, long-term construction, inclement weather conditions, or incidents. Since it is the base scenario in this sequence of example problems, no additional capacity adjustments are performed.

Step 5: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios are calculated from the demand flows in Exhibit 10-27 and from the segment capacities in Exhibit 10-28.

Exhibit 10-29
Segment Demand-to-Capacity Ratios for Example Problem 1

Time Step	Demand-to-Capacity Ratios by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.67	0.74	0.74	0.74	0.70	0.63	0.72	0.79	0.79	0.79	0.75
2	0.74	0.82	0.82	0.82	0.76	0.71	0.82	0.90	0.90	0.90	0.86
3	0.78	0.87	0.87	0.87	0.83	0.77	0.90	0.99	0.99	0.99	0.92
4	0.70	0.75	0.75	0.75	0.71	0.61	0.71	0.78	0.78	0.78	0.74
5	0.56	0.59	0.59	0.59	0.55	0.47	0.56	0.60	0.60	0.60	0.58

The computed demand-to-capacity ratio matrix in Exhibit 10-29 shows no segments with a v_d/c ratio greater than 1.0 in any time interval. Consequently, the facility is categorized as *globally undersaturated* and the analysis proceeds with computing the undersaturated service measures in Step 6a. Further, it is expected that no queuing will occur on the facility and that the volume served in each segment is identical to the input demand flows. Consequently, the matrix of volume-to-capacity ratios would be identical to the demand-to-capacity ratios in Exhibit 10-29. The resulting matrix of volumes served by segment and time interval is shown in Exhibit 10-30.

Exhibit 10-30
Volume-Served Matrix for Example Problem 1

Time Step	Volumes Served (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	4,505	4,955	4,955	4,955	4,685	5,225	4,865	5,315	5,315	5,315	5,045
2	4,955	5,495	5,495	5,495	5,135	5,855	5,495	6,035	6,035	6,035	5,765
3	5,225	5,855	5,855	5,855	5,585	6,395	6,035	6,665	6,665	6,665	6,215
4	4,685	5,045	5,045	5,045	4,775	5,135	4,775	5,225	5,225	5,225	4,955
5	3,785	3,965	3,965	3,965	3,695	3,965	3,785	4,055	4,055	4,055	3,875

Step 6a: Compute Undersaturated Segment Service Measures

Since the facility is globally undersaturated, the methodology proceeds to calculate service measures for each segment and each time period, starting with the first segment in Time Step 1. The computational details for each segment type are exactly as described in Chapters 11, 12, and 13. The weaving methodology in Chapter 13 checks whether the weaving short length L_S is less than or equal to the maximum weaving length L_{max}. It is assumed that, for any time interval where L_S is longer than L_{max}, the weaving segment will operate as a basic freeway segment.

The basic performance measures computed for each segment and each time step are the segment speed (Exhibit 10-31), density (Exhibit 10-32), and LOS (Exhibit 10-33).

Exhibit 10-31
Speed Matrix for Example Problem 1

Time Step	Speed (mi/h) by Segment 1	2	3	4	5	6	7	8	9	10	11
1	60.0	53.9	59.7	56.1	60.0	48.0	59.9	53.4	53.4	56.0	59.7
2	59.8	53.2	58.6	55.8	59.6	46.7	58.6	52.2	52.2	55.6	57.5
3	59.4	52.5	57.1	55.7	58.3	46.1	56.1	50.6	50.6	55.2	55.0
4	60.0	53.8	59.7	56.1	60.0	49.7	60.0	53.5	53.5	56.0	59.8
5	60.0	54.9	59.8	56.3	60.0	52.5	60.0	54.8	54.8	56.5	60.0

Exhibit 10-32
Density Matrix for Example Problem 1

Time Step	Density (veh/mi/ln) by Segment 1	2	3	4	5	6	7	8	9	10	11
1	25.0	30.7	27.7	29.4	26.0	27.2	27.1	33.2	33.2	31.7	28.2
2	27.6	34.5	31.3	32.8	28.7	31.3	31.3	38.5	38.5	36.2	33.4
3	29.3	37.2	34.2	35.0	31.9	34.6	35.8	43.9	43.9	40.3	37.7
4	26.0	31.3	28.2	30.0	26.5	25.8	26.5	32.5	32.5	31.1	27.6
5	21.0	24.1	22.1	23.5	20.5	18.9	21.0	24.7	24.7	23.9	21.5

Exhibit 10-33
LOS Matrix for Example Problem 1

Time Step	LOS by Segment 1	2	3	4	5	6	7	8	9	10	11
1	C	C	D	C	D	C	D	D	D	D	D
2	D	D	D	D	D	D	D	D	D	D	D
3	D	D	D	D	D	E	E	E	E	E	E
4	D	C	D	D	D	C	D	D	D	D	D
5	C	C	C	C	C	B	C	C	C	C	C

Step 7: Compute Facility Service Measures and Determine LOS

In the final analysis step, facilitywide performance and service measures are calculated for each time step. Example calculations are provided for the first time step only; summary results are shown for all five time steps.

First, the facility space mean speed S is calculated for time interval $t = 1$ from the 11 individual segment flows $SF(i, t)$, segment lengths $L(i)$, and space mean speeds in each segment and time period $U(i, t)$.

$$S(t=1) = \frac{\sum_{i=1}^{11} SF(i,1) \times L(i)}{\sum_{i=1}^{11} SF(i,1) \times \frac{L(i)}{U(i,1)}}$$

$$\begin{aligned}\sum_{i=1}^{11} SF(i,1) \times L(i) &= 4{,}505 \times 5{,}280 + 4{,}955 \times 1{,}500 + 4{,}955 \times 2{,}280 + 4{,}955 \times 1{,}500 \\ &+ 4{,}685 \times 5{,}280 + 5{,}225 \times 2{,}640 + 4{,}865 \times 5{,}280 + 5{,}315 \times 1{,}140 \\ &+ 5{,}315 \times 360 + 5{,}315 \times 1{,}140 + 5{,}045 \times 5{,}280 \\ &= 154{,}836{,}000 \text{ veh-ft}\end{aligned}$$

$$\begin{aligned}\sum_{i=1}^{11} SF(i,1) \times \frac{L(i)}{U(i,1)} &= (4{,}505 \times 5{,}280/60.00) + (4{,}955 \times 1{,}500/53.9) \\ &+ (4{,}955 \times 2{,}280/59.70) + (4{,}955 \times 1{,}500/56.10) \\ &+ (4{,}685 \times 5{,}280/60.00) + (5{,}225 \times 2{,}640/48.00) \\ &+ (4{,}865 \times 5{,}280/59.90) + (5{,}315 \times 1{,}140/53.40) \\ &+ (5{,}315 \times 360/53.40) + (5{,}315 \times 1{,}140/56.00) \\ &+ (5{,}045 \times 5{,}280/59.70) \\ &= 2{,}688{,}024 \text{ veh-ft/mi/h}\end{aligned}$$

$$S(t=1) = \frac{154{,}836{,}000}{2{,}688{,}024} = 57.6 \text{ mi/h}$$

Second, the average facility density is calculated for Time Step 1 from the individual segment densities D, segment lengths L, and number of vehicles in each segment N:

$$D(t=1)=\frac{\sum_{i=1}^{11} D(i,1)\times L(i)\times N(i,1)}{\sum_{i=1}^{11} L(i)N(i,1)}$$

$\sum_{i=1}^{11} D(i,1)\times L(i)\times N(i,1)$ = (25.0×5,280×3) + (30.7×1,500×3) + (27.7×2,280×3) + (29.4×1,500×3) + (26.0×5,280×3) + (27.2×2,640×4) + (27.1×5,280×3) + (33.2×1,140×3) + (33.2×360×3) + (31.7×1,140×3) + (28.2×5,280×3)
= 2,687,957 (veh/mi/ln)(ln-ft)

$\sum_{i=1}^{11} L(i)N(i,1)$ = (5,280×3) + (1,500×3) + (2,280×3) + (1,500×3) + (5,280×3) + (2,640×4) + (5,280×3) + (1,140×3) + (360×3) + (1,140×3) + (5,280×3)
= 97,680 ln-ft

$$D(t=1)=\frac{2{,}687{,}957}{97{,}680}=27.5\text{ veh/mi/ln}$$

These calculations are repeated for all five time steps. The overall space mean speed across all time intervals is calculated as follows:

$$S(p=5)=\frac{\sum_{p=1}^{5}\sum_{i=1}^{11} SF(i,p)L(i)}{\sum_{p=1}^{5}\sum_{i=1}^{11} SF(i,p)\dfrac{L(i)}{U(i,p)}}$$

The overall average density across all time intervals is calculated as follows:

$$D(p=5)=\frac{\sum_{p=1}^{5}\sum_{i=1}^{11} D(i,p)\times L(i)\times N(i,p)}{\sum_{p=1}^{5}\sum_{i=1}^{11} L(i)N(i,p)}$$

The resulting performance and service measures for Time Steps 1–5 and the facility totals are shown in Exhibit 10-34. The LOS for each time interval is determined directly from the average density for each time interval by using Exhibit 10-7. No LOS is defined for the average across all time intervals.

Exhibit 10-34
Facility Performance Measure Summary for Example Problem 1

	Performance Measures		
Time Step	Space Mean Speed (mi/h)	Average Density (veh/mi/ln)	LOS
1	57.6	27.5	D
2	56.6	31.3	D
3	55.1	34.8	E
4	57.9	27.5	D
5	58.4	21.4	C
Total	**56.9**	**28.5**	**—**

Discussion

This facility turned out to be globally undersaturated. Consequently, the facility-aggregated performance measures could be calculated directly from the individual segment performance measures. An assessment of the segment service measures across the time–space domain can begin to highlight areas of potential congestion. Visually, this process can be facilitated by plotting the v_d/c, v_a/c, speed, or density matrices in contour plots.

EXAMPLE PROBLEM 2: EVALUATION OF AN OVERSATURATED FACILITY

The Facility

The facility used in Example Problem 2 is identical to the one in Example Problem 1, which is shown in Exhibit 10-25 and Exhibit 10-26.

The Facts

In addition to the information in Exhibit 10-25 and Exhibit 10-26, the following characteristics of the freeway facility are known:

Heavy vehicles	=	5% trucks, 0% RVs (all movements);
Driver population	=	regular commuters;
FFS	=	60 mi/h (all mainline segments);
Ramp *FFS*	=	40 mi/h (all ramps);
Acceleration lane length	=	500 ft (all ramps);
Deceleration lane length	=	500 ft (all ramps);
D_{jam}	=	190 pc/mi/ln;
c_{IFL}	=	2,300 pc/h/ln (for *FFS* = 60 mi/h);
L_s	=	1,640 ft (for Weaving Segment 6);
TRD	=	1.0 ramp/mi;
Terrain	=	level;
Analysis duration	=	75 min (divided into five 15-min time steps); and
Demand adjustment	=	+11% increase in demand volumes across all segments and time steps compared with Example Problem 1.

Comments

The facility and all geometric inputs are identical to Example Problem 1. The same general comments apply. The results of Example Problem 1 suggested a globally undersaturated facility, but some segments were close to their capacity (v_d/c ratios approaching 1.0). In the second example, a facilitywide demand increase of 11% is applied to all segments and all time periods. Consequently, it is expected that parts of the facility may become oversaturated and that queues may form on the facility.

Step 1: Input Data

The revised traffic demand inputs for all 11 segments and five analysis intervals are shown in Exhibit 10-35.

Exhibit 10-35
Demand Inputs for Example Problem 2

Time Step (15 min)	Entering Flow Rate (veh/h)	Ramp Flow Rates by Time Period (veh/h)						Exiting Flow Rate (veh/h)
		ONR1	ONR2*	ONR3	OFR1	OFR2	OFR3	
1	5,001	500	599 (56)	500	600	400	300	5,600
2	5,500	599	799 (111)	599	400	400	300	6,399
3	5,800	699	899 (167)	699	300	400	500	6,899
4	5,200	400	400 (89)	500	300	400	300	5,500
5	4,201	200	300 (56)	300	300	200	200	4,301

* Numbers in parentheses indicate ONR-2 to OFR-2 demand flow rates in Weaving Segment 6.

The values in Exhibit 10-35 represent the adjusted demand flows on the facility as determined from field observations or demand projections. The actual volume served in each segment will be determined during application of the methodology and is expected to be less downstream of a congested segment. The demand flows are given for the extended time–space domain, consistent with this chapter's methodology. Peaking occurs in the third 15-min period. Since inputs are in the form of 15-min observations, no peak hour factor adjustment is necessary. Additional geometric and traffic-related inputs are as specified in Exhibit 10-25 and the facts section of the problem statement.

Step 2: Demand Adjustments

The traffic flows in Exhibit 10-35 have already been given in the form of actual demands and no further demand adjustments are necessary.

Step 3: Compute Segment Capacities

Since no changes to segment geometry were made, the segment capacities for basic and ramp segments are consistent with Example Problem 1 and Exhibit 10-28. Capacities for weaving segments are a function of weaving flow patterns, and the increased demand flows resulted in slight changes as shown in Exhibit 10-36.

Exhibit 10-36
Segment Capacities for Example Problem 2

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1						8,253					
2						8,260					
3	6,732	6,732	6,732	6,732	6,732	8,303	6,732	6,732	6,732	6,732	6,732
4						8,382					
5						8,443					

Step 4: Adjust Segment Capacities

No capacity adjustments are made in this example.

Step 5: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios in Exhibit 10-37 are calculated from the demand flows in Exhibit 10-35 and from the segment capacities in Exhibit 10-36.

Exhibit 10-37
Segment Demand-to-Capacity Ratios for Example Problem 2

Time Step	Demand-to-Capacity Ratios by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.74	0.82	0.82	0.82	0.77	0.70	0.80	0.88	0.88	0.88	0.83
2	0.82	0.91	0.91	0.91	0.85	0.79	0.91	1.00	1.00	1.00	0.95
3	0.86	0.97	0.97	0.97	0.92	0.85	1.00	**1.10**	**1.10**	**1.10**	**1.02**
4	0.77	0.83	0.83	0.83	0.79	0.68	0.79	0.86	0.86	0.86	0.82
5	0.62	0.65	0.65	0.65	0.61	0.52	0.62	0.67	0.67	0.67	0.64

The computed v_d/c matrix in Exhibit 10-37 shows that Segments 8–11 now have v_d/c ratios greater than 1.0 (bold values). Consequently, the facility is categorized as *oversaturated* and the analysis proceeds with computing the oversaturated service measures in Step 6b. Further, it is expected that queuing will occur on the facility upstream of the congested segments and that the volume served in each segment downstream of the congested segments will be less than the demand. This residual demand will be served in later time intervals, provided that upstream demand drops and queues are allowed to clear.

Step 6b: Compute Oversaturated Segment Service Measures

The oversaturated computations apply to any segment with a v_d/c ratio greater than 1.0 as well as any segments upstream of those segments that experience queuing as a result of the bottleneck. All remaining segments are analyzed by using the individual segment methodologies of Chapters 11, 12, and 13, as applicable, with the caveat that volumes served may differ from demand flows.

Similar to Example Problem 1, the methodology calculates performance measures for each segment and each time period, starting with the first segment in Time Step 1. The computations are repeated for all segments for Time Steps 1 and 2 without encountering a segment with $v_d/c > 1.0$. Once the methodology enters Time Period 3 and Segment 8, the oversaturated computational module is invoked.

As the first active bottleneck, the v_a/c ratio for Segment 8 will be exactly 1.0 and will process traffic at its capacity. Consequently, demand for all downstream segments will be metered by that bottleneck. The unsatisfied demand is stored in upstream segments, which causes queuing in Segment 7 and perhaps further upstream segments depending on the level of excess demand. The rate of growth of the vehicle queue (wave speed) is estimated from shock wave theory, as discussed in detail in Chapter 25, Freeway Facilities: Supplemental. The performance measures (speed and density) of any segment with queuing are recomputed as discussed in Chapter 25, and the newly calculated values override the results from the segment-specific procedures.

Any unsatisfied demand is serviced in later time periods. As a result, volumes served in later time periods may be higher than the period demand flows. The resulting matrix of volumes served for Example Problem 2 is shown in Exhibit 10-38. The table emphasizes cells where volumes served are less than demand flows (in **bold)** and where volumes served are greater than demand flows (*italicized).*

Exhibit 10-38
Volume-Served Matrix for Example Problem 2

Time Step	Volumes Served (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	5,001	5,500	5,500	5,500	5,200	5,800	5,400	5,900	5,900	5,900	5,600
2	5,500	6,099	6,099	6,099	5,700	6,400	6,099	6,699	6,699	6,699	6,399
3	5,800	6,499	6,499	6,499	**6,111**	**6,625**	**6,032**	**6,732**	**6,732**	**6,732**	**6,277**
4	5,200	5,600	5,600	5,600	*5,389*	*6,173*	*5,967*	*6,466*	*6,466*	*6,466*	*6,121*
5	4,201	4,401	4,401	4,401	4,101	4,401	4,201	4,501	4,501	4,501	4,301

As a result of the bottleneck activation in Segment 8 in Time Period 3, queues form in upstream Segments 7, 6, and 5. The queuing is associated with reduced speeds and increased densities in those segments. Details on how these measures are calculated for oversaturated segments are given in Chapter 25. The results in this chapter were obtained from the FREEVAL-2010 engine. The resulting performance measures computed for each segment and time interval are the speed (Exhibit 10-39), density (Exhibit 10-40), and LOS (Exhibit 10-41).

Exhibit 10-39
Speed Matrix for Example Problem 2

Time Step	Speed (mi/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	59.8	53.1	58.6	55.9	59.4	46.8	58.9	52.5	52.5	55.7	58.2
2	58.6	52.1	55.7	55.5	57.8	45.4	55.7	50.5	50.5	55.3	53.8
3	57.4	51.0	53.0	55.4	53.6	28.2	34.8	50.2	50.2	55.1	54.6
4	59.4	53.0	58.2	55.8	49.9	39.2	53.9	51.2	51.2	55.3	55.6
5	60.0	54.5	59.7	56.2	60.0	51.7	60.0	54.4	54.4	56.3	60.0

Exhibit 10-40
Density Matrix for Example Problem 2

Time Step	Density (veh/mi/ln) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	27.9	34.5	31.3	32.8	29.2	31.0	30.6	37.5	37.5	35.3	32.1
2	31.3	39.0	36.5	36.7	32.9	35.8	36.5	44.2	44.2	40.4	39.7
3	33.7	42.5	40.9	39.1	38.0	**58.8**	**57.7**	44.7	44.7	40.7	38.3
4	29.2	35.2	32.1	33.4	36.0	39.4	36.9	42.1	42.1	38.9	36.7
5	23.3	26.9	24.6	26.1	22.8	21.3	23.3	27.6	27.6	26.6	23.9

Exhibit 10-41
Expanded LOS Matrix for Example Problem 2

Time Step	Density-Based LOS by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	D	D	D	D	D	D	D	D	D	D	D
2	D	D	E	D	D	E	E	E	E	E	E
3	D	D	E	D	E	**F**	**F**	E	E	E	E
4	D	D	D	D	E	E	E	D	D	D	E
5	C	C	C	C	C	C	C	C	C	C	C
Time Step	**Demand-Based LOS by Segment**										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1											
2											
3								**F**	**F**	**F**	**F**
4											
5											

The LOS table for oversaturated facilities (Exhibit 10-41) distinguishes between the conventional density-based LOS and a segment demand-based LOS. The density-based stratification strictly depends on the prevailing average density on each segment. Segments downstream of the bottleneck, whose capacities are greater than or equal to the bottleneck capacity, operate at LOS E (or better), even though their v_d/c ratios were greater than 1.0. The demand-based LOS identifies those segments with demand-to-capacity ratios exceeding 1.0 as if they had been evaluated in isolation (i.e., using methodologies of Chapters 11,

12, and 13). By contrasting the two parts of the LOS table, the analyst can develop an understanding of the metering effect of the bottleneck.

Step 7: Compute Facility Service Measures and Determine LOS

In the final analysis step, facilitywide performance and service measures are calculated for each time interval (Exhibit 10-42), consistent with Example Problem 1. Only summary results are shown in this case, since the computations have already been shown. The facility operates at LOS F in Time Period 3, since one or more individual segments have *d/c* ratios ≥ 1.0, even though the average facility density is below the LOS F threshold.

Exhibit 10-42
Facility Performance Measure Summary for Example Problem 2

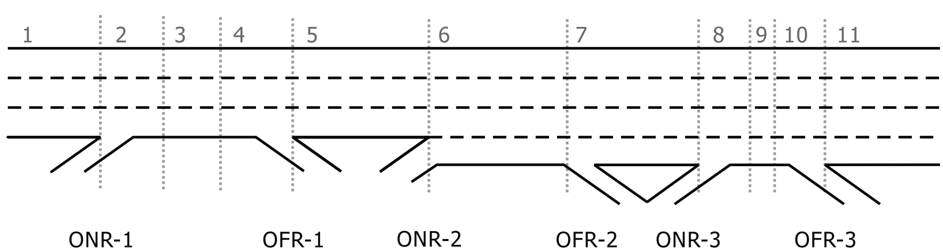
Time Interval	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	56.7	31.0	D
2	54.5	36.1	E
3	46.3	43.7	F
4	52.8	35.4	E
5	58.2	23.8	C
Total	**52.9**	**34.0**	**—**

EXAMPLE PROBLEM 3: CAPACITY IMPROVEMENTS TO AN OVERSATURATED FACILITY

The Facility

In this example, portions of the congested facility in Example Problem 2 are being improved in an attempt to alleviate the congestion resulting from the Segment 8 bottleneck. Exhibit 10-43 shows the upgraded facility geometry.

Exhibit 10-43
Freeway Facility in Example Problem 3



The modified geometry of the 6-mi directional freeway facility is reflected in Exhibit 10-44.

Exhibit 10-44
Geometry of Directional Freeway Facility in Example Problem 3

Segment No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	**4**	**4**	**4**	**4**	**4**

Note: B = basic freeway segment, W = weaving segment, ONR = on-ramp (merge) segment, OFR = off-ramp (diverge) segment, R = overlapping ramp segment.
Bold type indicates geometry changes from Example Problems 1 and 2.

The facility improvements consisted of adding a lane to Segments 7–11 to give the facility a continuous four-lane cross section starting in Segment 6. While the active bottleneck in Example Problem 2 was in Segment 8, the prior analysis showed that other segments (Segments 9–11) showed similar demand-to-capacity ratios greater than 1.0. Consequently, any capacity improvements that are limited to Segment 8 would have merely moved the spatial location of the bottleneck further downstream rather than improving the overall facility. Segments 9–11 may also be referred to as "hidden" or "inactive" bottlenecks, because their predicted congestion is mitigated by the upstream metering of traffic.

The Facts

In addition to the information contained in Exhibit 10-43 and Exhibit 10-44, the following characteristics of the freeway facility are known:

Heavy vehicles = 5% trucks, 0% RVs (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp FFS = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for FFS = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level;

Analysis duration = 75 min (divided into five 15-min intervals); and

Demand adjustment = +11% (all segments and all time intervals).

Comments

The traffic demand flow inputs are identical to those in Example Problem 2, which reflected an 11% increase in traffic applied to all segments and all time periods. In an attempt to solve the congestion effect found in the earlier example, the facility was widened in Segments 7 and 11. This change directly affects the capacities of those segments.

In a more subtle way, the proposed modifications also change some of the defining parameters of Weaving Segment 6 as well. With the added continuous lane downstream of the segment, the required number of lane changes from the ramp to the freeway is reduced from one to zero, following the guidelines in Chapter 12. These changes need to be considered when the undersaturated performance of that segment is evaluated. The weaving segment's capacity is unchanged relative to Example Problem 2, since, even with the proposed improvements, the number of weaving lanes remains two.

Step 1: Input Data

Traffic demand inputs for all 11 segments and five analysis intervals are identical to those in Example Problem 2 as shown in Exhibit 10-35. The values in Exhibit 10-35 represent the adjusted demand flows on the facility as determined from field observations or other sources. The actual volume served in each segment will be determined during the methodologies and is expected to be less downstream of a congested segment. Additional geometric and traffic-related inputs are as specified in Exhibit 10-44 and the facts section of the problem statement.

Step 2: Demand Adjustments

The traffic flows in Exhibit 10-35 have already been given in the form of actual demands and no further demand adjustments are necessary.

Step 3: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments. The resulting capacities are shown in Exhibit 10-45. Since the capacity of a weaving segment depends on traffic patterns, it varies by time period. The remaining capacities are constant for all five time steps. The capacities for Segments 1–5 and for Segments 7–11 are the same, since the segments have the same basic cross section.

Exhibit 10-45
Segment Capacities for Example Problem 3

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	6,732	6,732	6,732	6,732	6,732	8,253	8,976	8,976	8,976	8,976	8,976
2						8,260					
3						8,303					
4						8,382					
5						8,443					

Step 4: Adjust Segment Capacities

No additional capacity adjustments are made in this example.

Step 5: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios are calculated from the demand flows in Exhibit 10-35 and segment capacities in Exhibit 10-45.

Exhibit 10-46
Segment Demand-to-Capacity Ratios for Example Problem 3

Time Step	Demand-to-Capacity Ratio by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.74	0.82	0.82	0.82	0.77	0.70	0.60	0.66	0.66	0.66	0.62
2	0.82	0.91	0.91	0.91	0.85	0.79	0.68	0.75	0.75	0.75	0.71
3	0.86	0.97	0.97	0.97	0.92	0.85	0.75	0.82	0.82	0.82	0.77
4	0.77	0.83	0.83	0.83	0.79	0.68	0.59	0.65	0.65	0.65	0.61
5	0.62	0.65	0.65	0.65	0.61	0.52	0.47	0.50	0.50	0.50	0.48

The demand-to-capacity ratio matrix for Example Problem 3 (Exhibit 10-46) shows that the capacity improvements successfully reduced all the previously congested segments to $v_d/c < 1.0$. Therefore, it is expected that the facility will operate as *globally undersaturated* and that all segment performance measures can

be directly computed by using the methodologies in Chapters 11, 12, and 13 in Step 6a.

Step 6a: Compute Undersaturated Segment Service Measures

Since the facility is globally undersaturated, the methodology proceeds to calculate performance and service measures for each segment and each time step, starting with the first segment in Time Interval 1. The computational details for each segment type are exactly as described in Chapters 11, 12, and 13. The weaving methodology in Chapter 13 checks whether the weaving short length L_S is less than or equal to the maximum weaving length L_{max}. It is assumed that, for any time interval where L_S is longer than L_{max}, the weaving segment will operate as a basic freeway segment.

The basic performance service measures computed for each segment and each time interval include the segment speed (Exhibit 10-47), density (Exhibit 10-48), and LOS (Exhibit 10-49).

Exhibit 10-47
Speed Matrix for Example Problem 3

Time Step	Speed (mi/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	59.8	53.1	58.6	55.9	59.4	50.4	60.0	54.9	54.9	58.1	60.0
2	58.6	52.1	55.7	55.5	57.8	50.0	60.0	54.3	54.3	57.7	60.0
3	57.4	51.0	53.0	55.4	55.1	49.7	59.8	53.6	53.6	57.2	59.5
4	59.4	53.0	58.2	55.8	59.2	50.7	60.0	55.0	55.0	58.1	60.0
5	60.0	54.5	59.7	56.2	60.0	53.4	60.0	55.9	55.9	58.8	60.0

Exhibit 10-48
Density Matrix for Example Problem 3

Time Step	Density (veh/mi/ln) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	27.9	34.5	31.3	32.8	29.2	28.8	22.5	26.9	26.9	25.4	23.3
2	31.3	39.0	36.5	36.7	32.9	32.5	25.4	30.9	30.9	29.0	26.7
3	33.7	42.5	40.9	39.1	37.5	35.7	28.0	34.5	34.5	32.4	29.0
4	29.2	35.2	32.1	33.4	29.8	28.1	22.1	26.4	26.4	24.9	22.9
5	23.3	26.9	24.6	26.1	22.8	20.6	17.5	20.1	20.1	19.1	17.9

Exhibit 10-49
LOS Matrix for Example Problem 3

Time Step	LOS for Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	D	D	D	D	D	D	C	C	C	C	C
2	D	D	E	D	D	D	D	C	C	C	D
3	D	D	E	D	E	E	D	D	D	D	D
4	D	D	D	D	D	D	C	C	C	C	C
5	C	C	C	C	C	C	B	B	B	B	C

Step 7: Compute Facility Service Measures and Determine LOS

In the final analysis step, facilitywide performance and service measures are calculated for each time step (Exhibit 10-50), consistent with Example Problem 2. Only summary results are shown in this case, since the computations have already been shown. The improvement has been able to restore the facility LOS to the values experienced in the original pregrowth scenario shown in Exhibit 10-34.

Exhibit 10-50
Facility Performance Measure Summary for Example Problem 3

Time Step	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	57.9	26.8	D
2	57.1	30.4	D
3	56.0	33.5	D
4	57.8	26.9	D
5	58.6	20.8	C
Total	**57.3**	**27.7**	**—**

5. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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CHAPTER 11
BASIC FREEWAY SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Basic freeway segments are defined as those freeway segments that are outside the influence of merging, diverging, or weaving maneuvers. In general, this means that lane-changing activity is not significantly influenced by the presence of ramps and weaving segments. Lane-changing activity primarily reflects the normal desire of drivers to optimize their efficiency through lane-changing and passing maneuvers.

A complete discussion of influence areas is included in Chapter 10, Freeway Facilities, with additional discussion in Chapters 12, Freeway Weaving Segments, and 13, Freeway Merge and Diverge Segments. In general terms, the influence area of merge (on-ramp) segments extends for 1,500 ft downstream of the merge point; the influence area of diverge (off-ramp) segments extends for 1,500 ft upstream of the diverge point; and the influence area of weaving segments extends 500 ft upstream and downstream of the segment itself. This description is not to suggest that the influence of these segments cannot extend over a broader range, particularly under breakdown conditions. Under stable operations, however, these distances define the areas most affected by merge, diverge, and weaving movements. The impact of breakdowns in any type of freeway segment on adjacent segments can be addressed by using the methodology of Chapter 10, Freeway Facilities.

Chapter 11, Basic Freeway Segments, provides a methodology for analyzing the capacity and level of service (LOS) of existing or planned basic freeway segments. The methodology can also be used for design applications, where the number of lanes needed to provide a target LOS for an existing or projected demand flow rate can be found.

Analysis segments must have uniform geometric and traffic conditions, including demand flow rates.

Such analyses are applied to basic freeway segments with uniform characteristics. Uniform segments must have the same geometric and traffic characteristics, including a constant demand flow rate.

BASE CONDITIONS

Base conditions include good weather and visibility and no incidents or accidents. These conditions are always assumed to exist.

The base conditions under which the full capacity of a basic freeway segment is achieved include good weather, good visibility, no incidents or accidents, no work zone activity, and no pavement deterioration serious enough to affect operations. This chapter's methodology assumes that these conditions exist. If any of these conditions do not exist, the speed, LOS, and capacity of the freeway segment can be expected to be worse than those predicted by this methodology.

Base conditions also include the following conditions, which can be adjusted as the methodology is applied to address situations in which these conditions do not exist:

Base conditions also include 0% heavy vehicles, a driver population composed of regular users of the freeway, and 12-ft lane widths and minimum 6-ft right-side clearances.

The methodology provides adjustments for situations when these conditions do not apply.

- No heavy vehicles [trucks, buses, recreational vehicles (RVs)] in the traffic stream;
- A driver population composed primarily of regular users who are familiar with the facility; and
- Minimum 12-ft lane widths and 6-ft right-side clearances.

FLOW CHARACTERISTICS UNDER BASE CONDITIONS

Traffic flow within basic freeway segments can be highly varied depending on the conditions constricting flow at upstream and downstream bottleneck locations. Such bottlenecks can be created by merging, diverging, or weaving traffic; lane drops; maintenance and construction activities; traffic accidents or incidents; objects in the roadway; or all of the foregoing. Bottlenecks can exist even when a lane is not fully blocked. Partial blockages will cause drivers to slow and divert their paths. In addition, the practice of rubbernecking near roadside incidents or accidents can cause functional bottlenecks.

Types of Flow

Chapter 2 describes in more detail the types of traffic flow on basic freeway segments.

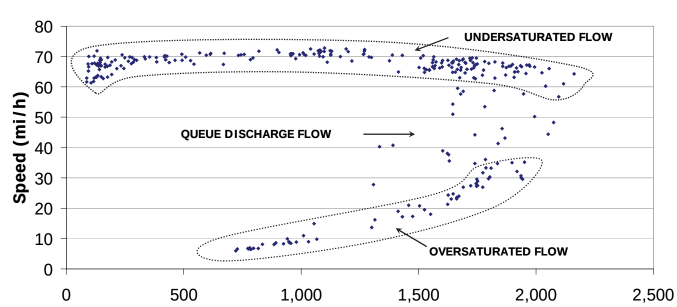
As was discussed in more detail in Chapter 2, Applications, traffic flow within a basic freeway segment can be categorized as one of three general types: undersaturated, queue discharge, and oversaturated.

- *Undersaturated flow* represents conditions under which the traffic stream is unaffected by upstream or downstream bottlenecks.
- *Queue discharge flow* represents traffic flow that has just passed through a bottleneck and is accelerating back to drivers' desired speeds for the prevailing conditions. As long as another downstream bottleneck does not exist, queue discharge flow is relatively stable until the queue is fully discharged.
- *Oversaturated flow* represents the conditions within a queue that has backed up from a downstream bottleneck. These flow conditions do not reflect the prevailing conditions of the site itself, but rather the consequences of a downstream problem. All oversaturated flow is considered to be congested.

An example of each of the three types of flow discussed is illustrated in Exhibit 11-1, using data from a freeway in California.

Exhibit 11-1
Three Types of Freeway Flow

LIVE GRAPH
Click here to view



Note: I-405, Los Angeles, Calif.
Source: California Department of Transportation, 2008.

The analysis methodology for basic freeway segments is based entirely on calibrations of the speed–flow relationships under base conditions with undersaturated flow. The methodology identifies cases in which failure has occurred but does not attempt to describe operating conditions when a segment has failed. The methodology of Chapter 10, Freeway Facilities, should be used for oversaturated conditions.

The basic freeway segment methodology is based on undersaturated flow conditions.

Speed–Flow Curves for Base Conditions

A set of speed–flow curves for basic freeway segments operating under base conditions is shown in Exhibit 11-2. There are five curves, one for each of five levels of free-flow speed (FFS): 75 mi/h, 70 mi/h, 65 mi/h, 60 mi/h, and 55 mi/h. Technically speaking, the FFS is the speed at the y-intercept of each curve. In practical terms, there are two ranges in the shape of the curves:

- For each curve, a range of flows exists from 0 pc/h/ln to a breakpoint in which speed remains constant at the FFS. The ranges vary for each of the curves as follows:

  FFS = 75 mi/h: 0–1,000 pc/h/ln;

  FFS = 70 mi/h: 0–1,200 pc/h/ln;

  FFS = 65 mi/h: 0–1,400 pc/h/ln;

  FFS = 60 mi/h: 0–1,600 pc/h/ln;

  FFS = 55 mi/h: 0–1,800 pc/h/ln.

- At flow rates above the breakpoint of each curve, speeds decline at an increasing rate until capacity is reached.

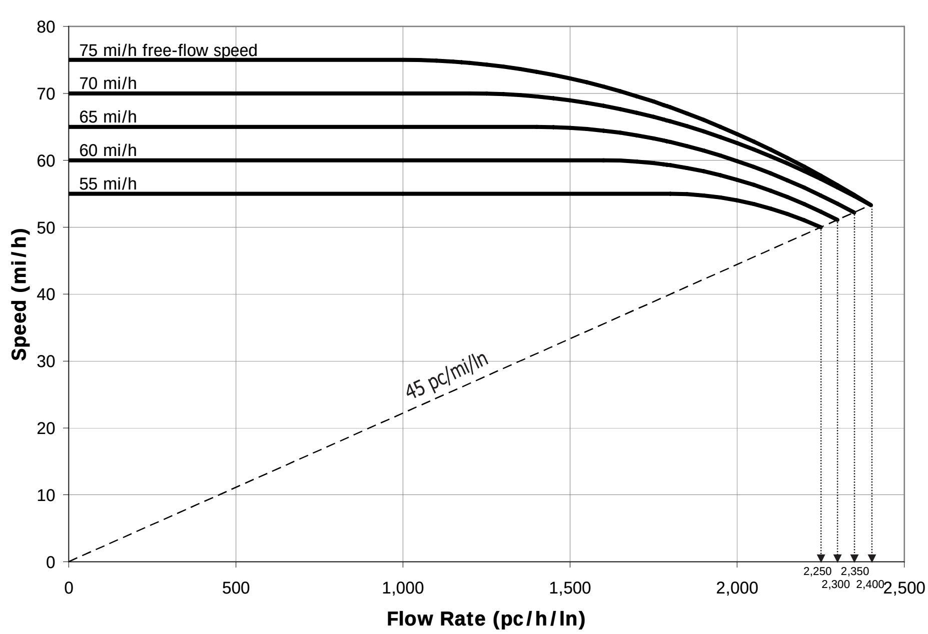


Exhibit 11-2
Speed–Flow Curves for Basic Freeway Segments Under Base Conditions

LIVE GRAPH
Click here to view

Exhibit 11-3 shows the equations that define each of the curves in Exhibit 11-2. Because estimating or measuring FFS is difficult, and there is considerable variation in observed and predicted values, no attempt should be made to

FFS should be rounded to the nearest 5 mi/h.

interpolate between the basic curves. FFS should be rounded to the nearest 5 mi/h as follows:

- ≥72.5 mi/h < 77.5 mi/h: use FFS = 75 mi/h,
- ≥67.5 mi/h < 72.5 mi/h: use FFS = 70 mi/h,
- ≥62.5 mi/h < 67.5 mi/h: use FFS = 65 mi/h,
- ≥57.5 mi/h < 62.5 mi/h: use FFS = 60 mi/h,
- ≥52.5 mi/h < 57.5 mi/h: use FFS = 55 mi/h.

Exhibit 11-3
Equations Describing Speed–Flow Curves in Exhibit 11-2 (Speeds in mi/h)

FFS (mi/h)	Breakpoint (pc/h/ln)	Flow Rate Range	
		≥0 ≤ Breakpoint	>Breakpoint ≤ Capacity
75	1,000	75	$75 - 0.00001107\ (v_p - 1{,}000)^2$
70	1,200	70	$70 - 0.00001160\ (v_p - 1{,}200)^2$
65	1,400	65	$65 - 0.00001418\ (v_p - 1{,}400)^2$
60	1,600	60	$60 - 0.00001816\ (v_p - 1{,}600)^2$
55	1,800	55	$55 - 0.00002469\ (v_p - 1{,}800)^2$

Notes: FFS = free-flow speed, v_p = demand flow rate (pc/h/ln) under equivalent base conditions.
Maximum flow rate for the equations is capacity: 2,400 pc/h/ln for 70- and 75-mph FFS; 2,350 pc/h/ln for 65-mph FFS; 2,300 pc/h/ln for 60-mph FFS; and 2,250 pc/h/ln for 55-mph FFS.

The research leading to these curves (*1, 2*) found that several factors affect the FFS of a basic freeway segment, including the lane width, right-shoulder clearance, and ramp density. Ramp density is the average number of on-ramps plus off-ramps in a 6-mi range, 3 mi upstream and 3 mi downstream of the midpoint of the study segment. Many other factors are likely to influence FFS: horizontal and vertical alignment, posted speed limits, level of speed enforcement, lighting conditions, and weather. Although these factors may affect FFS, little information is available that would allow their quantification.

CAPACITY UNDER BASE CONDITIONS

The capacity of a basic freeway segment under base conditions varies with the FFS. For 70- and 75-mi/h FFS, the capacity is 2,400 pc/h/ln. For lesser levels of FFS, capacity diminishes slightly. For 65-mi/h FFS, the capacity is 2,350 pc/h/ln; for 60-mi/h FFS, 2,300 pc/h/ln; and for 55-mi/h FFS, 2,250 pc/h/ln.

Base capacity values refer to the average flow rate across all lanes. Individual lanes could have stable flows in excess of these values.

Since freeways usually do not operate under base conditions, observed capacity values will typically be lower than the base capacity values.

Chapter 10, Freeway Facilities, contains information that would allow these values to be reduced to reflect long- and short-term construction and maintenance activities, adverse weather conditions, and accidents or incidents.

These values represent national norms. It should be remembered that capacity varies stochastically and that any given location could have a larger or smaller value. It should also be remembered that capacity refers to the *average flow rate across all lanes*. Thus, a three-lane basic freeway segment with a 70-mi/h FFS would have an expected base capacity of 3 × 2,400 = 7,200 pc/h. This flow would not be uniformly distributed across all lanes. Thus, one or two lanes could have stable base flows in excess of 2,400 pc/h/ln.

As shown in Exhibit 11-2, it is believed that basic freeway segments reach capacity at a density of approximately 45 passenger cars per mile per lane (pc/mi/ln), which may vary slightly from location to location. At this density,

vehicles are too closely spaced to dampen the impact of any perturbation in flow, such as a lane change or a vehicle entering the freeway, without causing a disruption that propagates upstream.

LOS FOR BASIC FREEWAY SEGMENTS

LOS for basic freeway segments is defined by density.

LOS on a basic freeway segment is defined by density. Although speed is a major concern of drivers as related to service quality, it would be difficult to describe LOS by using speed, since it remains constant up to flow rates of 1,000 to 1,800 pc/h/ln, depending on the FFS. Density describes the proximity to other vehicles and is related to the freedom to maneuver within the traffic stream. Unlike speed, however, density is sensitive to flow rates throughout the range of flows.

Exhibit 11-4 visually demonstrates the six LOS defined for basic freeway segments. LOS are defined to represent reasonable ranges in the three critical flow variables: speed, density, and flow rate.

Exhibit 11-4
LOS Examples

LOS A | LOS B
LOS C | LOS D
LOS E | LOS F

Freeway LOS Described

LOS A describes free-flow operations. FFS prevails on the freeway, and vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream. The effects of incidents or point breakdowns are easily absorbed.

LOS B represents reasonably free-flow operations, and FFS on the freeway is maintained. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents and point breakdowns are still easily absorbed.

LOS C provides for flow with speeds near the FFS of the freeway. Freedom to maneuver within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service quality will be significant. Queues may be expected to form behind any significant blockages.

LOS D is the level at which speeds begin to decline with increasing flows, with density increasing more quickly. Freedom to maneuver within the traffic stream is seriously limited and drivers experience reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.

LOS E describes operation at capacity. Operations on the freeway at this level are highly volatile because there are virtually no usable gaps within the traffic stream, leaving little room to maneuver within the traffic stream. Any disruption to the traffic stream, such as vehicles entering from a ramp or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic flow. At capacity, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown and substantial queuing. The physical and psychological comfort afforded to drivers is poor.

LOS F describes breakdown, or unstable flow. Such conditions exist within queues forming behind bottlenecks. Breakdowns occur for a number of reasons:

- Traffic incidents can temporarily reduce the capacity of a short segment, so that the number of vehicles arriving at a point is greater than the number of vehicles that can move through it.
- Points of recurring congestion, such as merge or weaving segments and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles that can be discharged.
- In analyses using forecast volumes, the projected flow rate can exceed the estimated capacity of a given location.

Breakdown (LOS F) occurs whenever the demand-to-capacity ratio exceeds 1.00.

In all cases, breakdown occurs when the ratio of existing demand to actual capacity, or of forecast demand to estimated capacity, exceeds 1.00. Operations immediately downstream of, or even at, such a point, however, are generally at or near LOS E, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharging vehicles move away from the bottleneck.

LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point. In practical terms, the point of the breakdown has a v/c ratio greater than 1.00, and is also labeled LOS F, although actual operations at the breakdown point and immediately downstream may actually reflect LOS E conditions. Whenever queues due to a breakdown exist, they have the potential to extend upstream for considerable distances.

The effects of a breakdown may extend upstream for a considerable distance.

LOS Criteria

A basic freeway segment can be characterized by three performance measures: density in passenger cars per mile per lane (pc/mi/ln), space mean speed in miles per hour (mi/h), and the ratio of demand flow rate to capacity (v/c). Each of these measures is an indication of how well traffic is being accommodated by the basic freeway segment.

Because speed is constant through a broad range of flows and the v/c ratio is not directly discernible to road users (except at capacity), the service measure for basic freeway segments is density. Exhibit 11-5 shows the criteria.

Exhibit 11-5
LOS Criteria for Basic Freeway Segments

LOS	Density (pc/mi/ln)
A	≤11
B	>11–18
C	>18–26
D	>26–35
E	>35–45
F	Demand exceeds capacity >45

For all LOS, the density boundaries on basic freeway segments are the same as those for surface multilane highways, except at the LOS E–F boundary. Traffic characteristics are such that the maximum flow rates at any given LOS are lower on multilane highways than on similar basic freeway segments.

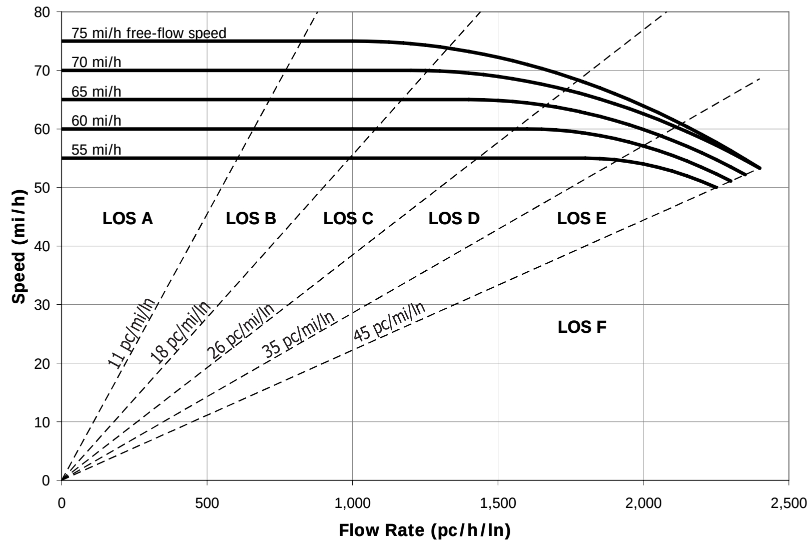
The specification of maximum densities for LOS A to D is based on the collective professional judgment of the members of the Transportation Research Board's Highway Capacity and Quality of Service Committee. The upper value shown for LOS F (45 pc/mi/ln) is the maximum density at which sustained flows at capacity are expected to occur. In effect, as indicated in the speed–flow curves of Exhibit 11-2, when a density of 45 pc/mi/ln is reached, flow is at capacity, and the v/c ratio is 1.00.

In the application of this chapter's methodology, however, LOS F is identified when demand exceeds capacity because the analytic methodology *does not allow* the determination of density when demand exceeds capacity. Although the density will be greater than 45 pc/h/ln, the methodology of Chapter 10, Freeway Facilities, must be applied to determine a more precise density for such cases.

Exhibit 11-6 illustrates the defined LOS on the base speed–flow curves. On a speed–flow plot, density is a line of constant slope beginning at the origin. The LOS boundaries were defined to produce reasonable ranges within each LOS on these speed–flow relationships.

Exhibit 11-6
LOS for Basic Freeway Segments

LIVE GRAPH
Click here to view



REQUIRED INPUT DATA

The analysis of a basic freeway segment requires details concerning the geometric characteristics of the segment and the demand characteristics of the users of the segment. This section presents the required input data for the basic freeway segment methodology; specifics about individual parameters are given in the Methodology section.

Freeway Data

The following information on the segment's geometric features is needed to conduct an analysis (typical ranges for these parameters are shown):

1. FFS: 55 to 75 mi/h;
2. Number of mainline freeway lanes (one direction): at least two;
3. Lane width: 10 ft to 12 ft or more;
4. Right-side lateral clearance: 0 ft to more than 6 ft;
5. Total ramp density: 0 to 6 ramps/mi; and
6. Terrain: level, rolling, or mountainous, or specific length and percent grade.

Demand Data

The following information on the segment's users is required:

1. Demand during the analysis hour or daily demand and *K*- and *D*-factors;
2. Heavy-vehicle presence (proportion of trucks, buses, and RVs): 0 to 100% in general terrain, or 0 to 25% or more for specific grades;
3. Peak-hour factor (PHF): up to 1.00; and
4. Driver population factor: 0.85 to 1.00.

Length of Analysis Period

The analysis period for any freeway analysis is generally the peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

2. METHODOLOGY

This chapter's methodology can be used to analyze the capacity, LOS, lane requirements, and effects of design features on the performance of basic freeway segments. The methodology is based on the results of an NCHRP study (*1*), which has been partially updated (*2*). A number of significant publications were also used in the development of the methodology (*3–12*).

LIMITATIONS OF METHODOLOGY

This chapter's methodology does not apply to or take into account (without modification by the analyst) the following:

- Special lanes reserved for a single vehicle type, such as high-occupancy-vehicle (HOV) lanes, truck lanes, and climbing lanes;
- Lane control (to restrict lane changing);
- Extended bridge and tunnel segments;
- Segments near a toll plaza;
- Facilities with FFS less than 55 mi/h or more than 75 mi/h;
- The influence of downstream queuing on a segment;
- Posted speed limit and enforcement practices;
- Presence of intelligent transportation systems (ITS) related to vehicle or driver guidance;
- Capacity-enhancing effects of ramp metering;
- Operational effects of oversaturated conditions; and
- Operational effects of construction operations.

In most of the cases just cited, the analyst would have to utilize alternative tools or draw on other research information and develop special-purpose modifications of this methodology to incorporate the effects of any of the cited conditions. Operational effects of oversaturated conditions, incidents, work zones, and weather and lighting conditions can be evaluated with the methodology of Chapter 10, Freeway Facilities. Operational effects of active traffic management measures are discussed in Chapter 35.

Active traffic management measures for freeways discussed in Chapter 35 consist of
- *Dynamic demand metering,*
- *Congestion pricing,*
- *Traveler information systems,*
- *Dynamic lane and shoulder management,*
- *Speed harmonization,*
- *Incident management, and*
- *Work zone traffic management.*

OVERVIEW OF METHODOLOGY

The methodology of this chapter is for the analysis of basic freeway segments. A method for analysis of extended lengths of freeway composed of a combination of basic freeway segments, weaving segments, and merge or diverge segments is found in Chapter 10, Freeway Facilities.

Exhibit 11-7 illustrates the basic methodology used in operational analysis. The methodology can also be directly applied to determine the number of lanes required to provide a target LOS for a given demand volume.

Exhibit 11-7
Overview of Operational Analysis Methodology for Basic Freeway Segments

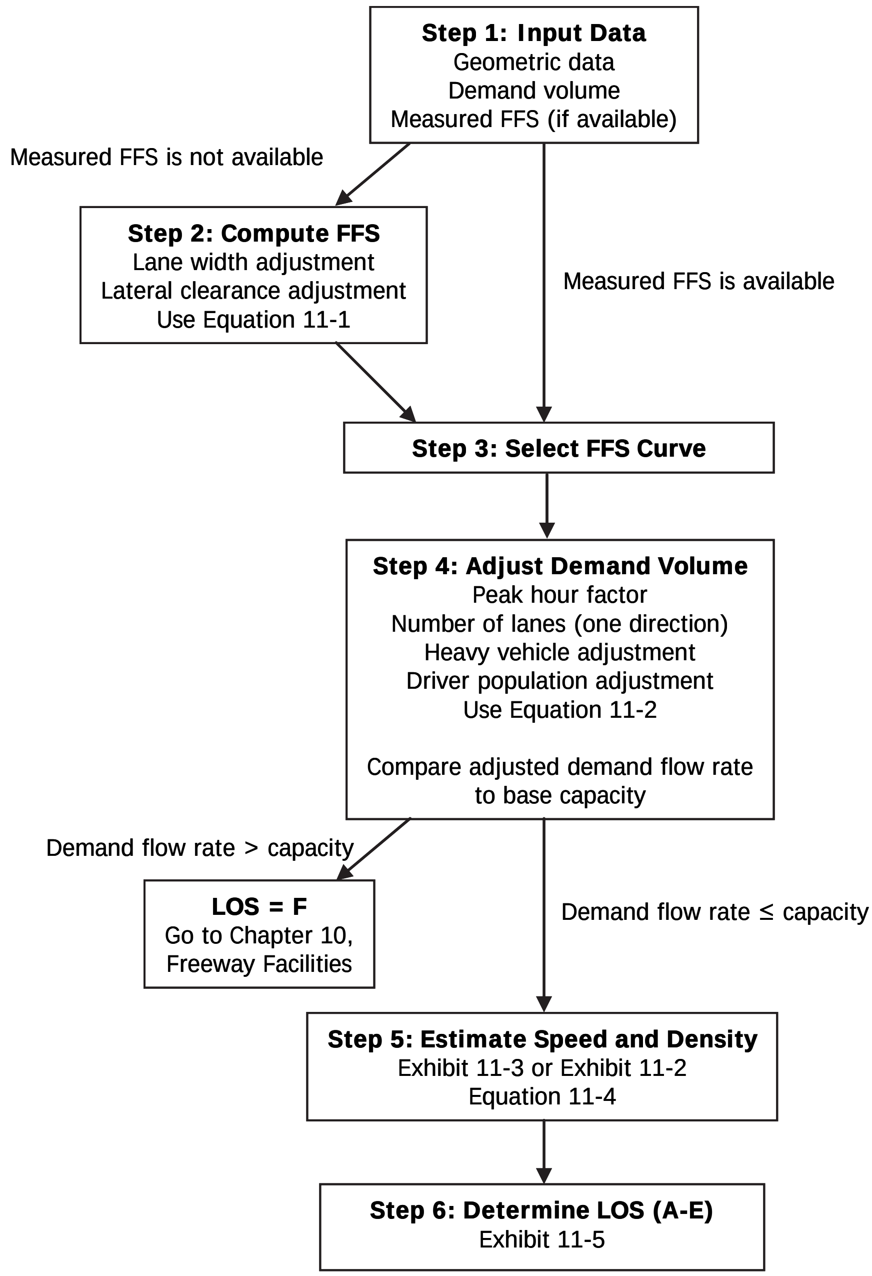


Exhibit 11-7 illustrates the methodology for operational analysis. Other types of analysis are described in the Applications section.

COMPUTATIONAL STEPS

Step 1: Input Data

For a typical operational analysis, as noted previously, the analyst would have to specify (with either site-specific or default values) demand volume, number and width of lanes, right-side lateral clearance, total ramp density, percent of heavy vehicles (trucks, buses, and RVs), PHF, terrain, and the driver population factor.

Step 2: Compute FFS

FFS can be determined directly from field measurements or can be estimated as described below.

Field Measurement of FFS

FFS is the mean speed of passenger cars during periods of low to moderate flow.

FFS is the mean speed of passenger cars measured during periods of low to moderate flow (up to 1,000 pc/h/ln). For a specific freeway segment, average speeds are virtually constant in this range of flow rates. If the FFS can be field measured, this is the preferable way to make the determination. If the FFS is measured directly, no adjustments are applied to the measured value.

The speed study should be conducted at a location that is representative of the segment at a time when flow rates are less than 1,000 pc/h/ln. The speed study should measure the speeds of all passenger cars or use a systematic sample (e.g., every tenth car in each lane). A sample of at least 100 passenger-car speeds should be obtained. Any speed measurement technique that has been found acceptable for other types of traffic engineering applications may be used. Further guidance on the conduct of speed studies is provided in standard traffic engineering publications, such as the Institute of Transportation Engineers *Manual of Traffic Engineering Studies* (*11*).

Estimating FFS

It is not possible to make field measurements for future facilities, and field measurement may not be possible or practical in all existing cases. In such cases, the segment's FFS may be estimated by using Equation 11-1, which is based on the physical characteristics of the segment under study:

Equation 11-1

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84}$$

where

FFS = FFS of basic freeway segment (mi/h),

f_{LW} = adjustment for lane width (mi/h),

f_{LC} = adjustment for right-side lateral clearance (mi/h), and

TRD = total ramp density (ramps/mi).

Base FFS

This methodology covers basic freeway segments with FFSs ranging from 55 mi/h to 75 mi/h. Thus, the predictive algorithm must start with a base speed of 75 mi/h or higher. A value of 75.4 mi/h was chosen, since it resulted in the most accurate predictions versus data collected in 2008.

Adjustment for Lane Width

The base condition for lane width is 12 ft or greater. When the average lane width across all lanes is less than 12 ft, the FFS is negatively affected. Adjustments to reflect the effect of narrower average lane width are shown in Exhibit 11-8.

Exhibit 11-8
Adjustment to FFS for Average Lane Width

Average Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Adjustment for Lateral Clearance

The base condition for right-side lateral clearance is 6 ft or greater. The lateral clearance is measured from the right edge of the travel lane to the nearest lateral obstruction. Care must be taken to identify a "lateral obstruction." Some obstructions may be continuous, such as retaining walls, concrete barriers, guardrails, or barrier curbs. Others may be periodic, such as light supports or bridge abutments. In some cases, drivers may become accustomed to certain types of obstructions, often making their influence on traffic negligible.

Exhibit 11-9 shows the adjustments to the base FFS due to the existence of obstructions closer than 6 ft to the right travel lane edge. Median clearances of 2 ft or more generally have little impact on traffic. No adjustments are available to reflect the presence of left-side lateral obstructions closer than 2 ft to the left travel lane edge. Such situations are, however, quite rare on modern freeways, except in constrained work zones.

Exhibit 11-9
Adjustment to FFS for Right-Side Lateral Clearance, f_{LC} (mi/h)

Right-Side Lateral Clearance (ft)	Lanes in One Direction			
	2	**3**	**4**	**≥5**
≥6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

The impact of a right-side lateral clearance restriction depends on both the distance to the obstruction and the number of lanes in one direction on the basic freeway segment. A lateral clearance restriction causes vehicles in the right lane to move somewhat to the left. This movement, in turn, affects vehicles in the next lane. As the number of lanes increases, the overall effect on freeway operations decreases.

Total Ramp Density

Equation 11-1 includes a term that accounts for the impact of total ramp density on FFS. Total ramp density is defined as the number of ramps (on and off, one direction) located between 3 mi upstream and 3 mi downstream of the midpoint of the basic freeway segment under study, divided by 6 mi. The total ramp density has been found to be a measure of the impact of merging and diverging vehicles on FFS.

Step 3: Select FFS Curve

As noted previously, once the FFS of the basic freeway segment is determined, one of the five base speed–flow curves (Exhibit 11-2) is selected for use in the analysis. Interpolation between curves is not recommended. Criteria for selecting an appropriate curve were given in the text following Exhibit 11-2.

Step 4: Adjust Demand Volume

Since the basic speed–flow curves of Exhibit 11-2 are based on flow rates in equivalent passenger cars per hour, with the driver population dominated by

regular users of the basic freeway segment, demand volumes expressed as vehicles per hour under prevailing conditions must be converted to this basis. Equation 11-2 is used for this adjustment:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

Equation 11-2

where

v_p = demand flow rate under equivalent base conditions (pc/h/ln),

V = demand volume under prevailing conditions (veh/h),

PHF = peak-hour factor,

N = number of lanes in analysis direction,

f_{HV} = adjustment factor for presence of heavy vehicles in traffic stream, and

f_p = adjustment factor for unfamiliar driver populations.

Peak Hour Factor

The PHF represents the variation in traffic flow within an hour. Observations of traffic flow consistently indicate that the flow rates found in the peak 15 min within an hour are not sustained throughout the entire hour. The application of the PHF in Equation 11-2 accounts for this phenomenon.

On freeways, typical PHFs range from 0.85 to 0.98 (*13*). Lower values within that range are typical of lower-volume conditions. Higher values within that range are typical of urban and suburban peak-hour conditions. Field data should be used if possible to develop PHFs that represent local conditions.

Adjustment for Heavy Vehicles

A heavy vehicle is defined as any vehicle with more than four wheels on the ground during normal operation. Such vehicles are generally categorized as trucks, buses, or RVs. Trucks cover a wide variety of vehicles, from single-unit trucks with double rear tires to triple-unit tractor–trailer combinations. Small panel or pickup trucks with only four wheels are, however, classified as passenger cars. Buses include intercity buses, public transit buses, and school buses. Because buses are in many ways similar to single-unit trucks, both types of vehicles are considered in one category. RVs include a wide variety of vehicles from self-contained motor homes to cars and small trucks with trailers (for boats, all-terrain vehicles, or other conveyances). It should be noted that most sport-utility vehicles have only four wheels and are thus categorized as passenger cars. The heavy-vehicle adjustment factor f_{HV} is computed as follows:

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right) + P_R\left(E_R - 1\right)}$$

Equation 11-3

where

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of trucks and buses in traffic stream,

P_R = proportion of RVs in traffic stream,

E_T = passenger-car equivalent (PCE) of one truck or bus in traffic stream, and

E_R = PCE of one RV in traffic stream.

The adjustment factor is found in a two-step process. First, the PCE for each truck or bus and RV is found for the prevailing conditions under study. These equivalency values represent the number of passenger cars that would use the same amount of freeway capacity as one truck or bus or RV under the prevailing conditions. Second, Equation 11-3 is used to convert the PCE values to the adjustment factor.

In many cases, trucks will be the only heavy-vehicle type present in the traffic stream. In others, the percentage of RVs will be small in comparison with trucks and buses. If the ratio of trucks and buses to RVs is 5:1 or greater, all heavy vehicles may be (but do not have to be) considered to be trucks.

The effect of heavy vehicles on traffic flow depends on terrain and grade conditions as well as traffic composition. PCEs can be selected for one of three conditions:

- Extended freeway segments in general terrain,
- Specific upgrades, or
- Specific downgrades.

Each of these conditions is more precisely defined and discussed next.

Equivalents for General Terrain Segments

General terrain refers to extended lengths of freeway containing a number of upgrades and downgrades where no one grade is long enough or steep enough to have a significant impact on the operation of the overall segment. As a guideline for this determination, extended segment analysis can be applied where grades are ≤2% and ≤0.25 mi long, or where grades between 2% and 3% are ≤0.50 mi long. For this determination, each upgrade and downgrade is considered to be a single grade, even if the grade is not uniform. The total length of the upgrade or downgrade is used with the steepest grade it contains. There are three categories of general terrain:

- *Level terrain*: Any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars. This type of terrain typically contains short grades of no more than 2%.
- *Rolling terrain*: Any combination of grades and horizontal or vertical alignment that causes heavy vehicles to reduce their speed substantially below those of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length

of time or at frequent intervals. *Crawl speed* is the maximum sustained speed that trucks can maintain on an extended upgrade of a given percent. If the grade is long enough, trucks will be forced to decelerate to the crawl speed, which they can maintain for extended distances. Appendix A contains truck-performance curves illustrating crawl speed and length of grade.

- *Mountainous terrain*: Any combination of grades and horizontal and vertical alignment that causes heavy vehicles to operate at crawl speed for significant distances or at frequent intervals.

The mountainous terrain category is rarely used, because individual grades will typically be longer, steeper, or both, than the criteria for general terrain analysis.

Mountainous terrain is relatively rare. Generally, in segments severe enough to cause the type of operation described for mountainous terrain, individual grades will be longer or steeper, or both, than the criteria for general terrain analysis.

Exhibit 11-10 shows PCEs for trucks and buses and RVs in general terrain segments.

Exhibit 11-10
PCEs for Heavy Vehicles in General Terrain Segments

Vehicle	PCE by Type of Terrain: Level	Rolling	Mountainous
Trucks and buses, E_T	1.5	2.5	4.5
RVs, E_R	1.2	2.0	4.0

Equivalents for Specific Upgrades

Any freeway grade between 2% and 3% and longer than 0.5 mi or 3% or greater and longer than 0.25 mi should be considered a separate segment. The analysis of such segments must consider the upgrade conditions and the downgrade conditions separately, as well as whether the grade is a single, isolated grade of constant percentage or part of a series forming a composite grade. The analysis of composite grades is discussed in Appendix A.

Several studies have shown that freeway truck populations have an average weight-to-horsepower ratio between 125 and 150 lb/hp. This methodology adopts PCEs that are calibrated for a mix of trucks and buses in this range. RVs vary considerably in both type and characteristics and include everything from cars with trailers to self-contained mobile campers. In addition to the variability of vehicle characteristics, RV drivers are typically not professionals, and their degree of skill in handling such vehicles also varies widely. Typical RV weight-to-horsepower ratios range from 30 to 60 lb/hp.

Exhibit 11-11 and Exhibit 11-12 give values of E_T for trucks and buses and E_R for RVs, respectively. These factors vary with the percent of grade, length of grade, and the proportion of heavy vehicles in the traffic stream. Maximum values occur when there are only a few heavy vehicles in the traffic stream. The equivalents decrease as the number of heavy vehicles increases because these vehicles tend to form platoons. Because heavy vehicles have more uniform operating characteristics, fewer large gaps are created in the traffic stream when they platoon, and the impact of a single heavy vehicle in a platoon is less severe than that of a single heavy vehicle in a stream of primarily passenger cars. The aggregate impact of heavy vehicles on the traffic stream, however, increases as numbers and percentages of heavy vehicles increase.

The grade length should include 25% of the length of the vertical curves at the start and end of the grade.

With two consecutive upgrades, 50% of the length of the vertical curve joining them should be included.

The point of interest is usually the spot where heavy vehicles would have the greatest impact on operations: the top of a grade, the top of the steepest grade in a series, or a ramp junction, for example.

The length of the grade is generally taken from a highway profile. It typically includes the straight portion of the grade plus some portion of the vertical curves at the beginning and end of the grade. It is recommended that 25% of the length of the vertical curves at both ends of the grade be included in the length. Where two consecutive upgrades are present, 50% of the length of the vertical curve joining them is included in the length of each grade.

In the analysis of upgrades, the point of interest is generally at the end of the grade, where heavy vehicles would have the maximum effect on operations. However, if a ramp junction is being analyzed, for example, the length of the grade to the merge or diverge point would be used.

On composite grades, the relative steepness of segments is important. If a 5% upgrade is followed by a 2% upgrade, for example, the maximum impact of heavy vehicles is most likely at the end of the 5% segment. Heavy vehicles would be expected to accelerate after entering the 2% segment.

Exhibit 11-11
PCEs for Trucks and Buses (E_T) on Upgrades

Upgrade (%)	Length (mi)	Proportion of Trucks and Buses								
		2%	4%	5%	6%	8%	10%	15%	20%	≥25%
≤2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
>2–3	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.50–0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.75–1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>1.00–1.50	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>1.50	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
>3–4	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50–0.75	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	>0.75–1.00	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	>1.00–1.50	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	>1.50	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
>4–5	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.50–0.75	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	>0.75–1.00	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	>1.00	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
>5–6	0.00–0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.30	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.30–0.50	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	>0.50–0.75	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	>0.75–1.00	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	>1.00	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
>6	0.00–0.25	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	1.0
	>0.25–0.30	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	>0.30–0.50	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	>0.50–0.75	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	>0.75–1.00	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	>1.00	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

Note: Interpolation for percentage of trucks and buses is recommended to the nearest 0.1.

Exhibit 11-12
PCEs for RVs (E_R) on Upgrades

Upgrade (%)	Length (mi)	Proportion of RVs 2%	4%	5%	6%	8%	10%	15%	20%	≥25%
≤2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
>2–3	0.00–0.50	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.50	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
>3–4	0.00–0.25	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.25–0.50	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
>4–5	0.00–0.25	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	>0.50	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
>5	0.00–0.25	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	>0.25–0.50	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	>0.50	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0

Note: Interpolation for percentage of RVs is recommended to the nearest 0.1.

Equivalents for Specific Downgrades

Knowledge of specific impacts of heavy vehicles on operating conditions on downgrades is limited. In general, if the downgrade is not severe enough to cause trucks to shift into a lower gear (to engage engine braking), heavy vehicles may be treated as if they were on level terrain segments. Where a downgrade is severe, trucks must often use low gears to avoid gaining too much speed and running out of control. In such cases, their effect on operating conditions is more significant than on level terrain. Exhibit 11-13 gives values of E_T for this situation.

Exhibit 11-13
PCEs for Trucks and Buses (E_T) on Specific Downgrades

Downgrade (%)	Length of Grade (mi)	Proportion of Trucks and Buses 5%	10%	15%	≥20%
<4	All	1.5	1.5	1.5	1.5
4–5	≤4	1.5	1.5	1.5	1.5
	>4	2.0	2.0	2.0	1.5
>5–6	≤4	1.5	1.5	1.5	1.5
	>4	5.5	4.0	4.0	3.0
>6	≤4	1.5	1.5	1.5	1.5
	>4	7.5	6.0	5.5	4.5

On downgrades, RVs are always treated as if they were on level terrain; E_R is therefore always 1.2 on downgrades regardless of the length or severity of the downgrade or the percentage of RVs in the traffic stream.

E_R is always 1.2 on downgrades.

Equivalents for Composite Grades

The vertical alignment of most freeways results in a continuous series of grades. It is often necessary to determine the effect of a series of grades in succession. The most straightforward technique is to compute the average grade from the beginning of the composite grade to the point of interest. The average grade is defined as the total rise from the beginning of the composite grade to the point in question divided by the length of the grade (to the point of interest).

The average-grade technique is an acceptable approach for grades in which all subsections are less than 4% or the total length of the grade is less than 4,000 ft. For more severe composite grades, a detailed technique is presented in Appendix A. This technique uses vehicle performance curves and equivalent speeds to determine the equivalent simple grade for analysis.

The average grade can be used when all component grades are <4% or the total length of the grades is <4,000 ft.

Appendix A provides a method for addressing more severe composite grades.

Adjustment for Driver Population

The base traffic stream characteristics for basic freeway segments are representative of traffic streams composed primarily of commuters, or drivers who are familiar with the facility. It is generally accepted that traffic streams with different characteristics (e.g., recreational drivers) use freeways less efficiently. Although data are sparse and reported results vary substantially, significantly lower capacities have been reported on weekends, particularly in recreational areas. It may generally be assumed that the reduction in capacity (LOS E) extends to service flow rates and service volumes for other LOS as well.

An f_p-value of 1.00 should generally be used, reflective of drivers who are regular users of the freeway.

The adjustment factor f_p is used to reflect the effect of driver population. The values of f_p range from 0.85 to 1.00 in most cases, although lower values have been observed in isolated cases. In general, the analyst should use a value of 1.00, which reflects commuters or otherwise-accustomed drivers, unless there is sufficient evidence that a lower value should be used. Where greater accuracy is needed, comparative field studies of commuter and recreational traffic flow and speeds are recommended.

Does LOS F Exist?

At this point, the demand volume has been converted to a demand flow rate in passenger cars per hour per lane under equivalent base conditions. This demand rate must be compared with the base capacity of the basic freeway segment (2,400 pc/h/ln for FFS = 75 mi/h and 70 mi/h; 2,350 pc/h/ln for FFS = 65 mi/h; 2,300 pc/h/ln for FFS = 60 mi/h; 2,250 pc/h/ln for FFS = 55 mi/h).

If the demand exceeds capacity, the LOS is F, and a breakdown has been identified. To analyze the impacts of such a breakdown, the methodology of Chapter 10, Freeway Facilities, must be used. No further analysis using the methodology of the current chapter is possible.

If the demand is less than or equal to capacity, the analysis continues to Step 5.

Step 5: Estimate Speed and Density

At this point in the methodology, the following have been determined: (*a*) the FFS and appropriate FFS curve for use in the analysis, and (*b*) the demand flow rate expressed in passenger cars per hour per lane under equivalent base conditions. With this information, the estimated speed and density of the traffic stream may be determined.

With the equations specified in Exhibit 11-3, the expected mean speed of the traffic stream can be computed. A graphical solution with Exhibit 11-2 can also be performed.

With the estimated speed determined, Equation 11-4 is used to estimate the density of the traffic stream:

$$D = \frac{v_p}{S}$$

Equation 11-4

where

D = density (pc/mi/ln),

v_p = demand flow rate (pc/h/ln), and

S = mean speed of traffic stream under base conditions (mi/h).

As has been noted, Equation 11-4 is only used when the v_p/c is less than or equal to 1.00. All cases in which this ratio is greater than 1.00 are LOS F. In these cases, the speed S will be outside the range of Exhibit 11-3 and Exhibit 11-4, and no speed can be estimated.

Where LOS F exists, the analyst is urged to consult Chapter 10, Freeway Facilities, which allows an analysis of the time and spatial impacts of a breakdown, including its effects on upstream and downstream segments.

Step 6: Determine LOS

Exhibit 11-5 is entered with the density obtained from Equation 11-4 to determine the expected prevailing LOS.

SENSITIVITY OF RESULTS

The freeway FFS is most sensitive to the total ramp density.

The FFS of basic freeway segments is most sensitive to the total ramp density. Exhibit 11-14 illustrates the resulting FFS when total ramp density varies from 0 ramps/mi to 6 ramps/mi. Standard lane widths and right-side clearances are assumed. A freeway with 0 ramps/mi represents a case in which there are no ramps within 3 mi on either side of the study location. This situation occurs primarily in rural areas, where interchanges may be 10 or more miles apart. In rare cases, ramp densities in excess of 6 ramps/mi may exist, particularly in dense urban areas.

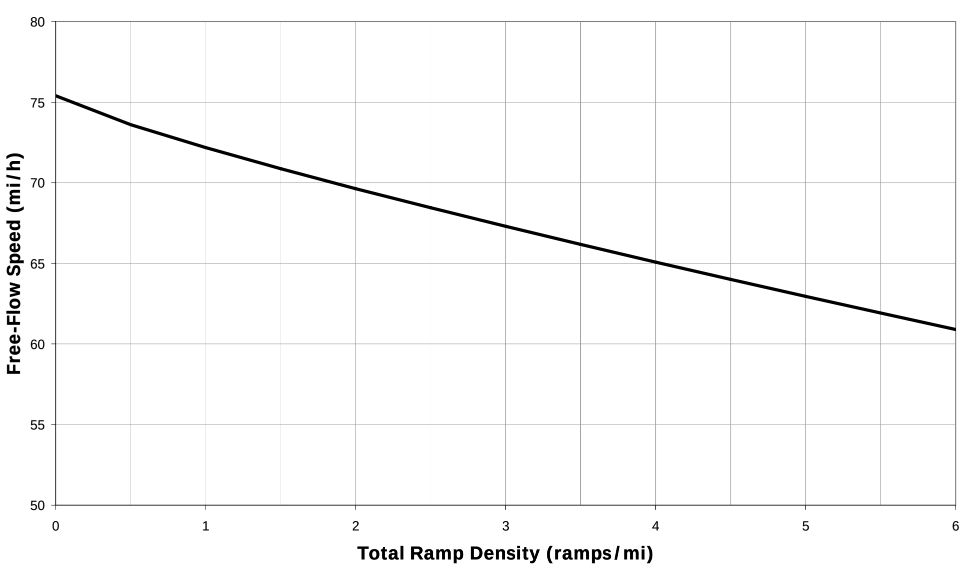


Exhibit 11-14
Sensitivity of FFS to Total Ramp Density

LIVE GRAPH
Click here to view

Each on- and off-ramp in the direction of travel is counted when total ramp density is determined.

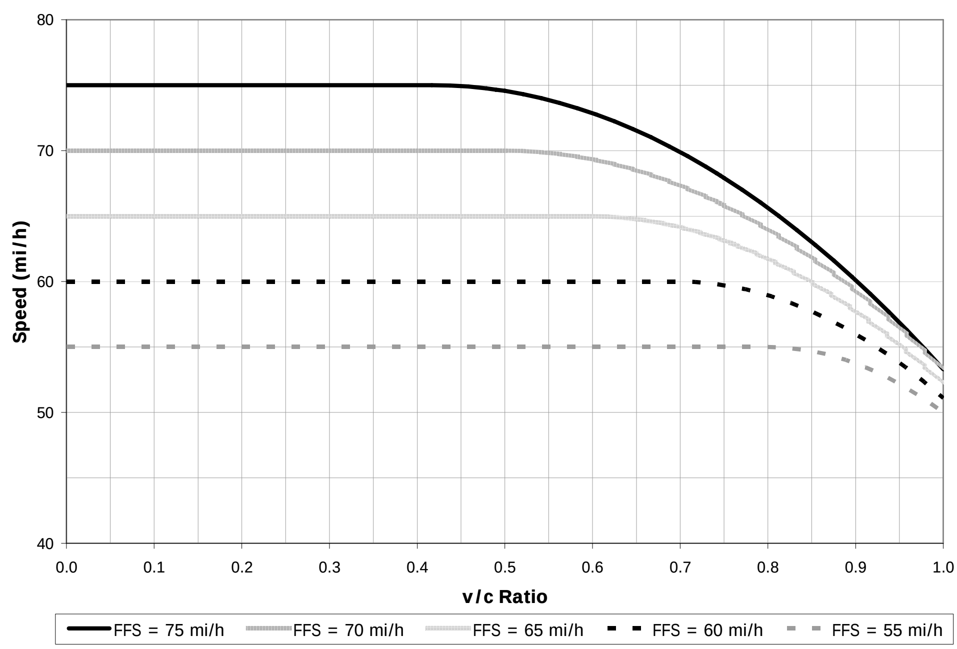
Higher total ramp densities represent suburban and urban situations as well as the type of interchanges present. Most interchanges involve two to four ramps. A full cloverleaf, for example, has four ramps: two on-ramps and two off-ramps in each direction. A diamond interchange has two ramps in each direction: one on-ramp and one off-ramp. Thus, a freeway with two cloverleaf interchanges fully contained within 1 mi would have a total ramp density of 8 ramps/mi. A freeway with two diamond interchanges fully contained within 1 mi would have a total density of 4 ramps/mi. This finding suggests that in any given situation (with comparable demand flows), cloverleaf interchanges will have a greater negative impact on FFS than diamond interchanges.

Although Exhibit 11-14 is not a straight line, the slope is relatively constant. On average, an increase of 2 ramps/mi in total ramp density causes a drop in FFS of approximately 5 mi/h. A reduction in FFS, of course, implies reductions in capacity and service volumes.

Exhibit 11-15 shows the relationship between speed and *v/c* ratio. Not unexpectedly, the shapes of these curves are similar to the basic speed–flow curves of Exhibit 11-2. Speed does not begin to decline until a *v/c* ratio of 0.42 to 0.80 is reached, depending on the FFS.

Exhibit 11-15
Speed Versus *v/c* Ratio

LIVE GRAPH
Click here to view



3. APPLICATIONS

The methodology in this chapter is relatively straightforward, so it can be directly used in any one of four applications:

1. *Operational analysis*: All traffic and roadway conditions are specified for an existing facility or a future facility with forecast conditions. The existing or expected LOS is determined.
2. *Design analysis*: A forecast demand volume is used, and key design parameters are specified (e.g., lane width and lateral clearance). The number of lanes required to deliver a target LOS is determined.
3. *Planning and preliminary engineering*: The basic scenario is the same as that for design analysis, except that the analysis is conducted at a much earlier stage of the development process. Inputs include default values, and the demand volume is usually stated as an annual average daily traffic (AADT) value.
4. *Service flow rates and service volumes*: The service flow rate, service volume, or daily service volume, or all three, are estimated for each LOS for an existing or future facility. All traffic and roadway conditions must be specified for this type of analysis.

Because the methodology and its algorithms are simple and do not involve iterations, all of the types of analysis cited can be done without the trial-and-error approach required by many other *Highway Capacity Manual* (HCM) methodologies.

DEFAULT VALUES

In using this chapter's methodology, a range of input data is needed. Most of these data should be field-measured or estimated values for the specific segment under consideration. When some of the data are not available, default values may be used. However, the use of default values will affect the accuracy of the output. Exhibit 11-16 shows the data that are required to conduct an operational analysis and the recommended default values when site-specific data are unavailable (*13*).

Exhibit 11-16
Required Input Data and Default Values for Basic Freeway Segments

Required Data	Default Values
Geometric Data	
Number of lanes in one direction	No default, must have site-specific value
Lane width (ft)	12 ft
Right-side lateral clearance (ft)	10 ft
Ramp density (ramps/mi)	No default, must have site-specific value
Terrain or specific grade (%, length)	No default, must have site-specific value
FFS (mi/h)	Urban, 70 mi/h; rural, 75 mi/h
Demand Data	
Length of analysis period (min)	15 min
PHF	0.94
Proportion of heavy vehicles (%)	Urban, 5%; rural, 12%*
Driver population factor	1.00

* Alternative state-specific default values for percentage of heavy vehicles are given in Chapter 26.

The analyst may also replace the default values of Exhibit 11-16 with defaults that have been locally calibrated.

Research into the percentage of heavy vehicles on uninterrupted-flow facilities (*13*) found a wide range of average values from state to state. Chapter 26 provides alternative default values for percentage of heavy vehicles by state and area population on the basis of data from the 2004 Highway Performance Monitoring System. Where states or local jurisdictions have developed their own values, these may be substituted. Analysts may also wish to develop their own default values based on more recent data.

ESTABLISH ANALYSIS BOUNDARIES

Determining capacity or LOS requires uniform traffic and roadway conditions on the analysis segment. Thus, any point where roadway or traffic conditions change must mark a boundary of the analysis segment.

At every ramp–freeway junction, the demand volume changes (as some vehicles enter or leave the traffic stream). Thus, any ramp junction should mark a boundary between adjacent basic freeway segments.

Ramp junctions, grade changes of 2% or more, changes in the freeway's geometric characteristics, and changes in speed limit are places where basic freeway segment boundaries should be established.

In addition to ramp–freeway junctions, the following conditions generally dictate that a boundary should be established between basic freeway segments:

- Change in the number of lanes (cross section),
- Changes in lane width or lateral clearance,
- Grade change of 2% or more on a specific or composite grade,
- Change in terrain category (for general terrain segments), or
- Change in posted speed limit.

The last is not directly involved in the analysis of a basic freeway segment but would probably reflect changes in ramp density or other freeway features.

TYPES OF ANALYSIS

Operational Analysis

Operational analyses find the expected LOS for specified roadway and traffic conditions.

The operational analysis application was fully specified in the Methodology section of this chapter. Operational analysis begins with all input parameters specified and is used to find the expected LOS that would result from the prevailing roadway and traffic conditions.

Design Analysis

Design analyses find the number of lanes required for a target LOS, given a specified demand volume.

In design analysis, a known demand volume is used to determine the number of lanes needed to deliver a target LOS. Two modifications are required to the operational analysis methodology. First, since the number of lanes is to be determined, the demand volume is converted to a demand flow rate in passenger cars per hour, not per lane, using Equation 11-5 instead of Equation 11-2:

Equation 11-5

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

where v is the demand flow rate in passenger cars per hour and all other variables are as previously defined.

Second, a maximum service flow rate for the target LOS is then selected from Exhibit 11-17. These values are selected from the base speed–flow curves of Exhibit 11-6 for each LOS.

Exhibit 11-17
Maximum Service Flow Rates in Passenger Cars per Hour per Lane for Basic Freeway Segments Under Base Conditions

FFS (mi/h)	Target Level of Service				
	A	B	C	D	E
75	820	1,310	1,750	2,110	2,400
70	770	1,250	1,690	2,080	2,400
65	710	1,170	1,630	2,030	2,350
60	660	1,080	1,560	2,010	2,300
55	600	990	1,430	1,900	2,250

Note: All values rounded to the nearest 10 pc/h/ln.

Next the number of lanes required to deliver the target LOS can be found from Equation 11-6:

Equation 11-6

$$N = \frac{v}{MSF_i}$$

where N is the number of lanes required and MSF_i is the maximum service flow rate for LOS i from Exhibit 11-17. Equation 11-5 and Equation 11-6 can be conveniently combined as Equation 11-7:

Equation 11-7

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

where all variables are as previously defined.

All fractional values of N must be rounded up.

The value of N resulting from Equation 11-6 or Equation 11-7 will most likely be fractional. Since only integer numbers of lanes can be constructed, the result is always rounded to the next-higher value. Thus, if the result is 3.2 lanes, 4 must be provided. The 3.2 lanes is, in effect, the minimum number of lanes needed to provide the target LOS. If the result were rounded to 3, a poorer LOS than the target value would result.

Because only whole lanes can be built, it may not be possible to achieve the target LOS for a given demand volume.

This rounding-up process will occasionally produce an interesting result: it is possible that a target LOS (for example, LOS C) cannot be achieved for a given demand volume. If 2.1 lanes are required to produce LOS C, providing 2 lanes would drop the LOS, most likely to D. However, if three lanes are provided, the LOS might actually improve to B. Thus, some judgment may be required to interpret the results. In this case, two lanes might be provided even though they would result in a borderline LOS D. Economic considerations might lead a decision maker to accept a slightly lower operating condition than that originally targeted.

Planning and Preliminary Engineering

Planning and preliminary engineering applications also find the number of lanes required to deliver a target LOS but provide more generalized input values to the methodology.

The objective of planning or preliminary engineering is to get a general idea of the number of lanes that will be required to deliver a target LOS. The primary differences are that many default values will be used and the demand volume will be usually expressed as an AADT. Thus, a planning and preliminary engineering analysis starts by converting the demand expressed as an AADT to

an estimate of the directional peak-hour demand volume (DDHV) with Equation 11-8:

Equation 11-8

$$V = DDHV = AADT \times K \times D$$

where K is the proportion of AADT occurring during the peak hour and D is the proportion of peak-hour volume traveling in the peak direction; all other variables are as previously defined.

Chapter 3 provides additional guidance on K- *and* D-*factors.*

On urban freeways, the typical range of K-factors is from 0.08 to 0.10. On rural freeways, values typically range between 0.09 and 0.13. Directional distributions also vary, as was illustrated in Chapter 3, Modal Characteristics, but a typical value for both urban and rural freeways is 0.55. As with all default values, locally or regionally calibrated values are preferred and yield more accurate results. Both the K-factor and the D-factor have a significant impact on the estimated hourly demand volume.

Once the hourly demand volume is estimated, the methodology follows the same path as that for design analysis.

Service Flow Rates, Service Volumes, and Daily Service Volumes

This chapter's methodology can be easily manipulated to produce service flow rates, service volumes, and daily service volumes for a basic freeway segment.

Exhibit 11-17 gave values of the maximum service flow rates, MSF_i, for each LOS for freeways of various FFSs. These values are given in terms of passenger cars per hour per lane under equivalent base conditions. A service flow rate, SF_i, is the maximum rate of flow that can exist while LOS i is maintained during the 15-min analysis period under prevailing conditions. It can be computed from the maximum service flow rate by using Equation 11-9:

Equation 11-9

$$SF_i = MSF_i \times N \times f_{HV} \times f_p$$

where all variables are as previously defined.

A service flow rate can be converted to a service volume, SV_i, by applying a PHF, as shown in Equation 11-10. A service volume is the maximum hourly volume that can exist while LOS i is maintained during the worst 15-min period of the analysis hour.

Equation 11-10

$$SV_i = SF_i \times PHF$$

where all variables are as previously defined.

A daily service volume, DSV_i, is the maximum AADT that can be accommodated by the facility under prevailing conditions while LOS i is maintained during the worst 15-min period of the analysis day. It is estimated from Equation 11-11:

Equation 11-11

$$DSV_i = \frac{SV_i}{K \times D}$$

where all variables are as previously defined.

Service flow rates *SF* and service volumes *SV* are stated for a single direction of the freeway. Daily service volumes *DSV* are stated as total volumes in *both* directions of the freeway.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for the application of alternative tools to the analysis of basic freeway segments. Additional information on this topic may be found in Chapter 26, Basic Freeway Segments: Supplemental.

Strengths of HCM Procedure

The HCM methodology provides FFS as an output, incorporates geometric characteristics, provides explicit capacity estimates, and produces a single deterministic estimate of traffic density.

This chapter's procedures were developed on the basis of extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent a body of expert consensus.

Specific strengths of the HCM basic freeway segment methodology include the following:

- It provides a detailed methodology for obtaining FFS. This methodology is based on various geometric characteristics. In simulation packages FFS (or an equivalent, such as desired speed) is an input.
- It considers geometric characteristics (such as lane widths), which are rarely, if ever, incorporated into simulation algorithms.
- It provides explicit capacity estimates. Simulation packages do not provide capacity estimates directly. Capacity estimates can only be obtained from simulators through multiple runs with oversaturated conditions. The user can modify simulated capacities by modifying specific input values such as the minimum acceptable headway.
- It produces a single deterministic estimate of traffic density, which is important for some purposes such as development impact review.

Limitations of HCM Procedures That Might Be Addressed by Alternative Tools

Deterministic models yield the same results for the same inputs each time they are implemented; stochastic models incorporate statistical variability. The same inputs yield different results in each use. For such models, an average result of X usages is employed as output.

Basic freeway segments can be analyzed by using a variety of stochastic and deterministic simulation packages that include freeways. These packages can be very useful in analyzing the extent of congestion when there are failures within the simulated facility range and when interaction with other freeway segments and other facilities is present.

Exhibit 11-18 tabulates the HCM limitations for basic freeway segments along with the potential for improved treatment by alternative tools.

Additional Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the capacity, speed, and density of a basic freeway segment, given the segment's traffic demand and characteristics. Alternative tools offer additional performance measures,

including delay, stops, queue lengths, fuel consumption, pollution, and operating costs.

Exhibit 11-18
Limitations of HCM Basic Freeway Segments Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Special lanes reserved for a single vehicle type, such as HOV, truck, and climbing lanes	Modeled explicitly by simulation
Extended bridge and tunnel segments	Can be approximated by using assumptions related to desired speed and number of lanes along each segment
Segments near a toll plaza	Can be approximated by using assumptions related to discharge at toll plaza
Facilities with FFS less than 55 mi/h or more than 75 mi/h	Modeled explicitly by simulation
Oversaturated conditions (refer to Chapters 10 and 26 for further discussion)	Modeled explicitly by simulation
Influence of downstream blockages or queuing on a segment	Modeled explicitly by simulation
Posted speed limit and extent of police enforcement	Can be approximated by using assumptions related to desired speed along a given segment
Presence of ITS features related to vehicle or driver guidance	Several features modeled explicitly by simulation; others may be approximated by using assumptions (for example, by modifying origin–destination demands by time interval)

As with most other procedural chapters in the HCM, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of downstream conditions on lane utilization and backup beyond the segment boundary is a good example of a situation that can benefit from the increased insight offered by a microscopic model.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS for basic freeway segments is based on traffic density expressed in passenger cars per mile per lane. The HCM methodology estimates density by dividing the flow rate by the average passenger-car speed. Simulation models typically estimate density by dividing the average number of vehicles in the segment by the area of the segment (in lane miles). The result is vehicles per lane mile. This measurement corresponds to density based on space mean speed. The HCM-reported density is also based on space mean speed, but because there is no variability in the speeds, the space mean speed is equal to the time mean speed. Generally, increased speed variability in driver behavior (which simulators usually include) results in lower average space mean speed and higher density.

In obtaining density from alternative models, it is important to take into account the following:

- The vehicles included in the density estimation (for example, whether only the vehicles that have exited the link are considered);
- The manner in which auxiliary lanes are considered;

- The units used for density, since a simulation package would typically provide density in units of vehicles rather than passenger cars; converting the simulation outputs to passenger cars with the HCM PCE values is typically not appropriate, given that the simulation should already account for the effects of heavy vehicles on a microscopic basis—with heavy vehicles operating at lower speeds and at longer headways—thus making any additional adjustments duplicative;
- The units used in the reporting of density (e.g., whether it is reported per lane mile);
- The homogeneity of the analysis segment, since the HCM does not use the segment length as an input (unless it is a specific upgrade or downgrade segment, where the length is used to estimate the PCE values), and conditions are assumed to be homogeneous for the entire segment; and
- The driver variability assumed in the simulation package, since increased driver variability will generally increase the average density.

Regarding capacity, the HCM provides capacity estimates in passenger cars per hour per lane as a function of FFS. To compare the HCM's estimates with capacity estimates from a simulation package, the following should be considered:

- The manner in which a simulation package provides the number of vehicles exiting a segment; in some cases it may be necessary to provide virtual detectors at a specific point on the simulated segment so that the maximum throughput can be obtained;
- The units used to specify maximum throughput, since a simulation package would do this in units of vehicles rather than passenger cars; converting these to passenger cars by using the HCM PCE values is typically not appropriate, since differences between automobile and heavy-vehicle performance should already be accounted for microscopically within a simulation; and
- The incorporation of other simulation inputs, such as the "minimum separation of vehicles," that affect the capacity result.

Conceptual Differences Between HCM and Simulation Modeling That Preclude Direct Comparison of Results

The HCM's methodology is based on the relationship between speed and flow for various values of FFS. One fundamental potential difference between the HCM and other models is this relationship. For example, the HCM assumes a constant speed for a broad range of flows. However, this is not necessarily the case for any given simulation package, some of which assume a continuously decreasing speed with increasing flow. Furthermore, in some simulation packages, that relationship changes when certain parameters are modified. Therefore, if performance measures are compatible between the HCM and an alternative model for a given set of flows, this will not necessarily be the case for all other sets of flows.

Adjustment of Simulation Parameters to HCM Results

The most important elements to be adjusted when a basic freeway segment is analyzed are the speed–flow relationship or the capacity, or both. The speed–flow relationship should be examined as a function of the given FFS. That FFS should match the field- or HCM-estimated value. Some tools only accept integer values of FFS, whereas the HCM may provide a fractional value as an intermediate calculation result.

Step-by-Step Recommendations for Applying Alternative Tools

This section provides recommendations specifically for freeway segments (general guidance on selecting and applying simulation packages is provided in Chapter 6, HCM and Alternative Analysis Tools). To apply an alternative tool to the analysis of basic freeway segments, the following steps should be taken:

1. Determine whether the chosen tool can provide density and capacity for a basic freeway segment and the approach used to obtain those values. Once the analyst is satisfied that density and capacity can be obtained and that values compatible with those of the HCM can also be obtained, proceed with the analysis.
2. Determine the FFS of the study site, either from field data or by estimating it according to this chapter's methodology.
3. Enter all available geometric and traffic characteristics into the simulation package and install virtual detectors along the study segment, if necessary, to obtain speeds and flows.
4. By loading the study network over capacity, obtain the maximum throughput and compare it with the HCM estimate. Calibrate the simulation package by modifying parameters related to the minimum time headway, so that the capacity obtained by the simulator closely matches the HCM estimate. Estimate the required number of runs to be conducted so that the comparison is statistically valid.
5. If the analysis requires evaluating various different demand conditions for the segment, plot the simulator's speed–flow curve and compare it with the HCM relationship. Attempt to calibrate the simulation package by modifying parameters related to driver behavior, such as the distribution of driver types. It is possible that the simulation cannot be calibrated to match the HCM speed–flow relationship. In that case, the results should be viewed with caution in terms of their compatibility with the HCM methods.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 26, in Volume 4 of the HCM, provides two supplemental problems that examine situations beyond the scope of this chapter's methodology by using a typical microsimulation-based tool. Both problems are based on Example Problem 3 (found in the next section of this chapter), which analyzes a six-lane freeway segment in a growing urban area. The first supplemental problem evaluates the facility when an HOV lane is added, and the second problem analyzes operations with an incident within the segment.

4. EXAMPLE PROBLEMS

Exhibit 11-19
List of Example Problems

Example Problem	Description	Application
1	Four-lane freeway LOS	Operational analysis
2	Number of lanes required for target LOS	Design analysis
3	Six-lane freeway LOS and capacity	Operational and planning analysis
4	LOS on upgrades and downgrades	Operational analysis
5	Design-hour volume and number of lanes	Planning analysis
6	Service flow rates and service volumes	Planning analysis

EXAMPLE PROBLEM 1: FOUR-LANE FREEWAY LOS

The Facts

- Four-lane freeway (two lanes in each direction);
- Lane width = 11 ft;
- Right-side lateral clearance = 2 ft;
- Commuter traffic (regular users);
- Peak-hour, peak-direction demand volume = 2,000 veh/h;
- Traffic composition: 5% trucks, 0% RVs;
- PHF = 0.92;
- One cloverleaf interchange per mile; and
- Rolling terrain.

Comments

The task is to find the expected LOS for this freeway during the worst 15 min of the peak hour. With one cloverleaf interchange per mile, the total ramp density will be 4 ramps/mi.

Step 1: Input Data

All input data are specified above.

Step 2: Compute FFS

The FFS of the freeway is estimated as follows:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84}$$

The adjustment for lane width is selected from Exhibit 11-8 for 11-ft lanes (1.9 mi/h). The adjustment for right-side lateral clearance is selected from Exhibit 11-9 for a 2-ft clearance on a freeway with two lanes in one direction (2.4 mi/h). The total ramp density is 4 ramps/mi. Then

$$FFS = 75.4 - 1.9 - 2.4 - 3.22\left(4^{0.84}\right) = 60.8 \text{ mi/h}$$

Step 3: Select FFS Curve

As the FFS calculated in Step 2 is greater than or equal to 57.5 and less than 62.5 mi/h, the 60-mi/h speed–flow curve will be used for this analysis.

Step 4: Adjust Demand Volume

The demand volume must be adjusted to a flow rate that reflects passenger cars per hour per lane under equivalent base conditions by using Equation 11-2:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

The demand volume is given as 2,000 veh/h. The PHF is specified to be 0.92, and there are two lanes in each direction. The driver population factor is 1.00, since regular users (commuters) are also specified. Trucks make up 5% of the traffic stream, so a heavy-vehicle adjustment factor must be determined.

From Exhibit 11-10, the PCE for trucks is 2.5 for rolling terrain. The heavy-vehicle adjustment factor is then computed by using Equation 11-3:

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right) + P_R\left(E_R - 1\right)}$$

$$f_{HV} = \frac{1}{1 + 0.05\left(2.5 - 1\right) + 0} = 0.930$$

Then

$$v_p = \frac{2{,}000}{0.92 \times 2 \times 0.93 \times 1.00} = 1{,}169 \text{ pc/h/ln}$$

Since this value is less than the base capacity of 2,300 pc/h/ln for a freeway with FFS = 60 mi/h, LOS F does not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

The FFS of the basic freeway segment is now estimated along with the demand flow rate in passenger cars per hour per lane under equivalent base conditions. From Exhibit 11-3, the equation for estimating the speed of the traffic stream is selected for a 60-mi/h FFS, with a flow rate less than 1,600 pc/h/ln. This is the constant-speed portion of the curve, so S = 60 mi/h. The density of the traffic stream may now be computed as

$$D = \frac{v_p}{S} = \frac{1{,}169}{60} = 19.5 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 11-5, a density of 19.5 pc/mi/ln corresponds to LOS C but is close to the boundary for LOS B, which is a maximum of 18 pc/mi/ln. This solution could also be calculated graphically by using Exhibit 11-6 as a base (Exhibit 11-20).

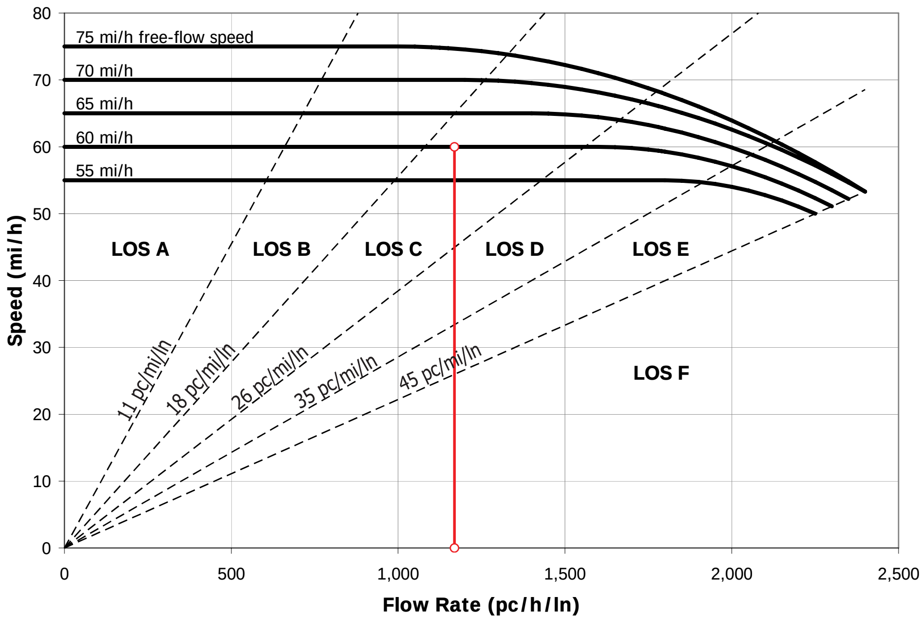


Exhibit 11-20
Graphical Solution for Example Problem 1

LIVE GRAPH
Click here to view

Discussion

This basic freeway segment of a four-lane freeway is expected to operate at LOS C during the worst 15 min of the peak hour. It is important to note that the operation, although at LOS C, is close to the LOS B boundary. In most jurisdictions, this operation would be considered to be quite acceptable; therefore, no remediation would normally be required.

EXAMPLE PROBLEM 2: NUMBER OF LANES REQUIRED FOR TARGET LOS

The Facts

- Demand volume = 4,000 veh/h (one direction);
- Level terrain;
- Traffic composition: 15% trucks, 3% RVs;
- Provision of 12-ft lanes;
- Provision of 6-ft right-side lateral clearance;
- Commuter traffic (regular users);
- PHF = 0.85;
- Ramp density = 3 ramps/mi; and
- Target LOS = D.

Comments

This is a classic design application of the methodology. The number of lanes needed to provide LOS D during the worst 15 min of the peak hour is to be determined.

Step 1: Input Data

All input data were specified previously.

Step 2: Compute FFS

The FFS is estimated by using Equation 11-1. Because the lane width and lateral clearance to be provided on the new freeway will be 12 ft and 6 ft, respectively, there are no adjustments for these features. The total ramp density is given as 3 ramps/mi. Then

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84}$$

$$FFS = 75.4 - 0.0 - 0.0 - 3.22\left(3^{0.84}\right) = 67.3 \text{ mi/h}$$

Step 3: Select FFS Curve

Since the FFS calculated in Step 2 is greater than or equal to 62.5 and less than 67.5 mi/h, the 65-mi/h speed–flow curve will be used for this analysis.

Step 4: Estimate Number of Lanes Needed

Because this is a design analysis, Step 4 of the operational analysis methodology is modified. Equation 11-7 may be used directly to determine the number of lanes needed to provide for at least LOS D:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

A value of the maximum service flow rate must be selected from Exhibit 11-17 for a FFS of 65 mi/h and LOS D. This value is 2,030 pc/h/ln. The PHF is given as 0.85. The driver population factor is 1.00, since commuters are involved. A heavy-vehicle factor for 15% trucks and 3% RVs must be determined by using Exhibit 11-10 for level terrain. The PCEs of trucks and RVs in level terrain are 1.5 and 1.2, respectively. Then

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right) + P_R\left(E_R - 1\right)}$$

$$f_{HV} = \frac{1}{1 + 0.15(1.5 - 1) + 0.03(1.2 - 1)} = 0.925$$

and

$$N = \frac{4{,}000}{2030 \times 0.85 \times 0.925 \times 1.00} = 2.51 \text{ lanes}$$

It is not possible to build 2.51 lanes. To provide a minimum of LOS D, it will be necessary to provide three lanes in each direction, or a six-lane freeway.

At this point, the design application ends. It is possible, however, to consider what speed, density, and LOS will prevail when three lanes are actually provided. Therefore, the example problem continues with Steps 5 and 6.

Step 5: Estimate Speed and Density

In pursuing additional information, the problem now reverts to an operational analysis of a three-lane basic freeway segment with a demand volume of 4,000 pc/h.

Equation 11-2 is used to compute the actual demand flow rate per lane under equivalent base conditions:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

$$v_p = \frac{4{,}000}{0.85 \times 3 \times 0.925 \times 1.00} = 1{,}696 \text{ pc/h/ln}$$

The expected speed of the traffic stream may be estimated either by using Exhibit 11-6 (for a graphical solution) or by selecting the appropriate equation from Exhibit 11-3—in this case, using FFS = 65 mi/h and a demand flow rate over 1,400 pc/h/ln. With the latter approach,

$$S = 65 - 0.00001418\left(v_p - 1{,}400\right)^2$$

$$S = 65 - 0.00001418\,(1{,}696 - 1{,}400)^2 = 63.8 \text{ mi/h}$$

The density may now be computed:

$$D = \frac{v_p}{S} = \frac{1{,}696}{63.8} = 26.6 \text{ pc/mi/ln}$$

Step 6: Determine LOS

Entering Exhibit 11-5 with a density of 26.6 pc/mi/ln, the LOS is D but is very close to the boundary of LOS C, which is 26 pc/mi/ln.

Discussion

The resulting LOS is D, which was the target for the design. Although the minimum number of lanes needed was 2.51, which would have provided for a minimal LOS D, providing three lanes yields a density that is close to the LOS C boundary. In any event, the target LOS of the design will be met by providing a six-lane basic freeway segment.

EXAMPLE PROBLEM 3: SIX-LANE FREEWAY LOS AND CAPACITY

The Facts

- Volume of 5,000 veh/h (one direction, existing);
- Volume of 5,600 veh/h (one direction, in 3 years);
- Traffic composition: 10% trucks, no RVs;
- Level terrain;
- Three lanes in each direction;
- FFS = 70 mi/h (measured);
- PHF = 0.95;
- Commuter traffic (regular users); and
- Traffic growth after 3 years = 4% per year.

Comments

This example consists of two operational analyses, one for the present demand volume of 5,000 pc/h and one for the demand volume of 5,600 pc/h expected in 3 years. In addition, a planning element is introduced: Assuming that traffic grows as expected, when will the capacity of the roadway be exceeded? This analysis requires that capacity be determined in addition to the normal output of operational analyses.

Step 1: Input Data

All input data were given previously.

Step 2: Compute FFS

Step 2 is not needed since a measured FFS is given (70 mi/h).

Step 3: Select FFS Curve

Step 3 is not needed. The FFS curve for 70 mi/h will be used, based on the measured value.

Step 4: Adjust Demand Volume

In this case, two demand volumes will be adjusted by using Equation 11-2:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

The PHF is given as 0.95, and there are three lanes in each direction. The driver population adjustment factor will be 1.00, for regular users. The heavy-vehicle factor must reflect 10% trucks in level terrain. From Exhibit 11-10, the PCE for trucks in level terrain is 1.5. Equation 11-3 then gives the following:

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right) + P_R\left(E_R - 1\right)}$$

$$f_{HV} = \frac{1}{1 + 0.10\left(1.5 - 1\right) + 0} = 0.952$$

Two values of v_p will be computed: one for present conditions and one for conditions in 3 years:

$$v_p\text{ (present)} = \frac{5{,}000}{0.95 \times 3 \times 0.952 \times 1.00} = 1{,}843 \text{ pc/h}$$

$$v_p\text{ (future)} = \frac{5{,}600}{0.95 \times 3 \times 0.952 \times 1.00} = 2{,}064 \text{ pc/h}$$

Step 5: Estimate Speed and Density

Two values of speed and density will be estimated, one each for the present and future conditions stated. The equations of Exhibit 11-3 will be used to estimate speeds. One equation applies to both cases, a 70-mi/h FFS with a flow rate over 1,200 pc/h/ln:

$$S(\text{present}) = 70 - 0.00001160\left(v_p - 1{,}200\right)^2$$

$$S(\text{present}) = 70 - 0.00001160(1{,}843 - 1{,}200)^2 = 65.2 \text{ mi/h}$$

$$S(\text{future}) = 70 - 0.00001160\left(v_p - 1{,}200\right)^2$$

$$S(\text{future}) = 70 - 0.00001160(2{,}064 - 1{,}200)^2 = 61.3 \text{ mi/h}$$

The corresponding densities may now be estimated as follows:

$$D = \frac{v_p}{S}$$

$$D(\text{present}) = \frac{1{,}843}{65.2} = 28.3 \text{ pc/mi/ln}$$

$$D(\text{future}) = \frac{2{,}064}{61.3} = 33.7 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 11-5, the LOS for the present situation is D, and the LOS for the future scenario (in 3 years) is also D, despite the increase in density.

Step 7: When Will Capacity Be Reached?

Step 7 is an additional step for this problem. To answer the question, the capacity of the basic freeway segment must be estimated. From Exhibit 11-17, the maximum service flow rate for LOS E on a basic freeway segment with a 70-mi/h FFS is 2,400 pc/h/ln. This flow rate is synonymous with capacity.

The analyst must be sure that the capacity and demand flow rates compared in Step 7 are on the same basis. The 2,400 pc/h/ln is a flow rate under equivalent base conditions. The demand flow rate in 3 years was estimated to be 2,064 pc/h/ln on this basis. These two values, therefore, may be compared. As an alternative, the capacity could be computed for prevailing conditions:

$$SF_E = MSF_E \times N \times f_{HV} \times f_p$$

$$SF_E = 2{,}400 \times 3 \times 0.952 \times 1.00 = 6{,}854 \text{ veh/h}$$

This capacity, however, is stated as a *flow rate*. The demand volume is stated as an hourly volume. Thus, a *service volume* for LOS E is needed:

$$SV_E = SF_E \times PHF = 6{,}854 \times 0.95 = 6{,}511 \text{ veh/h}$$

The problem may be solved either by comparing the demand volume of 5,600 veh/h (in 3 years) with the hourly capacity of 6,511 veh/h or by comparing the demand flow rate under equivalent base conditions of 2,064 pc/h/ln with the

base capacity of 2,400 pc/h/ln. With the hourly demand volume and hourly capacity,

$$6{,}511 = 5{,}600(1.04)^n$$

$$n = 3.85 \text{ years}$$

On the basis of the forecasts of traffic growth, the basic freeway segment described will reach capacity within 7 years (the demand of 5,600 veh/h occurs 3 years from the present).

Discussion

The LOS on this segment will remain D within 3 years despite the increase in density. The demand is expected to exceed capacity within 7 years. Given the normal lead times for planning, design, and approvals before the start of construction, it is probable that planning and preliminary design for an improvement should be started immediately.

EXAMPLE PROBLEM 4: LOS ON UPGRADES AND DOWNGRADES

The Facts

- Demand volume = 2,300 veh/h (one direction);
- Traffic composition: 15% trucks, no RVs;
- PHF = 0.90;
- FFS = 70 mi/h upgrade, 75 mi/h downgrade (measured);
- Unfamiliar drivers (f_p = 0.95); and
- Composite grade: 3,000 ft at 3%, followed by 2,600 ft at 5%.

Comments

This is a typical operational analysis. The expected outcome is an assessment of the LOS on both the upgrade and the downgrade. However, the problem deals with a specific grade and a composite grade. Because there is a segment of the grade that is greater than 4% and the total length of the composite grade exceeds 4,000 ft, the special procedure in Appendix A must be applied. That procedure will yield an equivalent constant-percent grade of 3,000 + 2,600 = 5,600 ft (1.06 mi), which has the same impact on heavy vehicles as the composite grade described.

Composite Grade

Exhibit 11-21 shows the conversion of the composite grade to a grade of constant percent 5,600 ft long. At the end of such a grade, the final speed of heavy vehicles is approximately the same as that on the composite grade.

A vertical line enters the truck performance curves at 3,000 ft extending to the +3% grade curve, indicating that the speed of trucks after 3,000 ft of +3% grade is approximately 42 mi/h. This is also the speed at which the truck enters the +5% grade; it corresponds to the same speed as that of a truck on a +5% grade after 1,300 ft. The truck travels another 2,600 ft (to 3,900 ft) on the +5% curve,

where a final speed of 27 mi/h is reached. The intersection of a horizontal drawn at 27 mi/h and a vertical drawn at a total length of grade of 5,600 ft yields the equivalent of +5%. In effect, because trucks on this grade are at crawl speed, it does not matter how long the grade is: 27 mi/h can be maintained indefinitely.

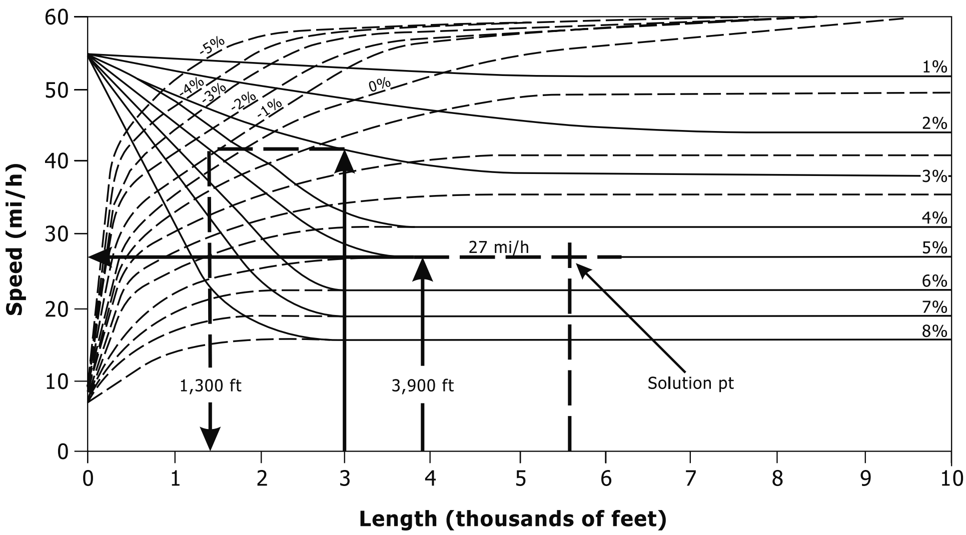


Exhibit 11-21
Determination of Composite Grade Equivalents for Example Problem 4

LIVE GRAPH
Click here to view

The equivalent grade is 5%, 5,600 ft. This equivalent should be applied to *both the upgrade and the downgrade*, even though it is developed specifically for the upgrade.

Although the truck acceleration curves of Appendix A could be used to develop a separate downgrade composite equivalent, it would be very misleading. The truck performance curves assume a maximum speed of 60 mi/h. On a long, steep downgrade, trucks will achieve much higher speeds.

It is highly likely that trucks will be forced to use a low gear to apply engine braking on the grade described. Thus, PCEs for the downgrade will be selected from Exhibit 11-13.

Step 1: Input Data

All input data were specified previously.

Step 2: Compute FFS

FFSs were measured in the field. The upgrade FFS is 70 mi/h; the downgrade FFS is 75 mi/h.

Step 3: Select FFS Curve

The 70-mi/h curve will be used for the upgrade; the 75-mi/h curve will be used for the downgrade.

Step 4: Adjust Demand Volume

The demand flow rates in passenger cars per hour per lane for the upgrade and downgrade are estimated by using Equation 11-2:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

The PHF is 0.90, there are two lanes on the upgrade and two lanes on the downgrade, and f_p is specified as 0.95. Heavy-vehicle adjustment factors, however, must be determined separately for the upgrade and the downgrade.

The PCE for trucks (E_T) on the upgrade is selected from Exhibit 11-11 for a grade of 5%, >1.00 mi long, with 15% trucks: 3.0. The PCE for the trucks on the downgrade is selected from Exhibit 11-13 for a grade of 4% to 5%, ≤4 mi long: 1.5.

The heavy-vehicle adjustment factors, f_{HV}, are computed by using Equation 11-3:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV}\text{ (upgrade)} = \frac{1}{1 + 0.15(3 - 1) + 0} = 0.769$$

$$f_{HV}\left(\text{downgrade}\right) = \frac{1}{1 + 0.15(1.5 - 1) + 0} = 0.930$$

Then

$$v_p\left(\text{upgrade}\right) = \frac{2{,}300}{0.90 \times 2 \times 0.769 \times 0.95} = 1{,}749\text{ pc/h/ln}$$

$$v_p\left(\text{downgrade}\right) = \frac{2{,}300}{0.90 \times 2 \times 0.930 \times 0.95} = 1{,}446\text{ pc/h/ln}$$

Since neither of these values exceeds the base capacity of a freeway with FFS = 75 mi/h (downgrade) or FFS = 70 mi/h (upgrade), LOS F does not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

With the FFS and the demand flow rate determined for both the upgrade and the downgrade, the expected speed and density on each may now be estimated. Speed is estimated by using the equations of Exhibit 11-3.

For the upgrade, the FFS is 70 mi/h, and the demand flow rate is greater than 1,200 pc/h/ln. Then

$$S = 70 - 0.00001160\left(v_p - 1{,}200\right)^2$$

$$S = 70 - 0.00001160\,(1{,}749 - 1{,}200)^2 = 66.5\text{ mi/h}$$

For the downgrade, the FFS is 75 mi/h, and the demand flow rate is greater than 1,000 pc/h/ln. Then

$$S = 75 - 0.00001107\left(v_p - 1{,}000\right)^2$$

$$S = 75 - 0.00001107\,(1{,}446 - 1{,}000)^2 = 72.8\text{ mi/h}$$

Densities may now be estimated from the demand flow rates and estimated speeds:

$$D = \frac{v_p}{S}$$

$$D(\text{upgrade}) = \frac{1{,}749}{66.5} = 26.3 \text{ pc/mi/ln}$$

$$D(\text{downgrade}) = \frac{1{,}446}{72.8} = 19.9 \text{ pc/mi/ln}$$

Step 6: Determine LOS

As shown in Exhibit 11-5, the upgrade LOS is D; the downgrade LOS is C. Both levels, however, are close to the boundaries for better operations—the upgrade is close to the boundary for LOS C (D = 26 pc/mi/ln) and the downgrade is close to the boundary for LOS B (D = 18 pc/mi/ln).

Discussion

Both the upgrade and the downgrade are operating at what would generally be called acceptable levels. If traffic grows over time, the addition of a truck climbing lane on the upgrade might be considered.

EXAMPLE PROBLEM 5: DESIGN-HOUR VOLUME AND NUMBER OF LANES

The Facts

- Demand volume = 75,000 veh/day,
- Proportion of AADT in the peak hour: 0.09,
- Directional distribution: 55/45,
- Rolling terrain, and
- Target LOS = D.

Comments

In this planning and preliminary engineering application, several input variables are not specified, so default values will have to be used. With knowledge of local conditions and freeway design standards, the following default values will be used in the solution: FFS = 65 mi/h; 5% trucks, no RVs; PHF = 0.95; and f_p = 1.00.

Determining Opening-Day Directional Design-Hour Volume

Because the demand volume is given as an AADT, it must be converted to a directional design-hour volume (DDHV) by using Equation 11-8:

$$V = DDHV = AADT \times K \times D$$

$$V = DDHV = 75{,}000 \times 0.09 \times 0.55 = 3{,}713 \text{ veh/h}$$

Step 1: Input Data

All input data were specified.

Step 2: Compute FFS

A default value of 65 mi/h will be used in this problem.

Step 3: Select FFS Curve

The 65-mi/h speed–flow curve will be used in this problem.

Step 4: Determine Number of Lanes Required

After estimating the demand volume on an hourly basis, the remainder of this solution follows the design application. The number of lanes needed is estimated by using Equation 11-7:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

The maximum service flow rate is selected from Exhibit 11-17 for LOS D on a 65-mi/h basic freeway segment: 2,030 pc/h/ln. The PHF is a default value: 0.95. The driver population factor is also a default value: 1.00. The freeway is in rolling terrain and is expected to have 5% trucks (another default value). From Equation 11-10, for rolling terrain, $E_T = 2.5$. Then

$$f_{HV} = \frac{1}{1+0.05(2.5-1)+0} = 0.930$$

$$N = \frac{3{,}713}{2{,}030 \times 0.95 \times 0.93 \times 1.00} = 2.07 \text{ lanes}$$

Because fractional lanes cannot be built, three lanes will have to be provided in each direction to ensure that LOS D is provided during the worst 15 min of the peak hour. Therefore, the resulting LOS may be better than the design target.

Step 5: Estimate Speed and Density

In order to determine the likely LOS resulting from a six-lane freeway, the speed and density should be estimated. Equation 11-2 is used to determine the actual demand flow rate for three lanes:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

$$v_p = \frac{3{,}713}{0.95 \times 3 \times 0.93 \times 1.00} = 1{,}401 \text{ pc/h/ln}$$

From Exhibit 11-3, for a 65-mi/h basic freeway segment with more than 1,400 pc/h/ln, the expected speed is

$$S = 65 - 0.00001418\left(v_p - 1{,}400\right)^2$$

$$S = 65 - 0.00001418(1{,}401 - 1{,}400)^2 = 65.0 \text{ mi/h}$$

and the density is

$$D = \frac{v_p}{S} = \frac{1,401}{65.0} = 21.6 \text{ pc/mi/ln}$$

Step 6: Determine LOS

As shown in Exhibit 11-5, the expected LOS is C.

Discussion

This problem illustrates an interesting point: given the parameters of this example problem, the target LOS of D cannot be achieved on opening day. If a four-lane freeway (two lanes in each direction) is built, LOS E will result. If a six-lane freeway (three lanes in each direction) is built, LOS C will result.

EXAMPLE PROBLEM 6: SERVICE FLOW RATES AND SERVICE VOLUMES

The Facts

- Eight-lane freeway;
- FFS = 70 mi/h (measured);
- Traffic composition: 8% trucks, 1% RVs;
- Rolling terrain;
- PHF = 0.87;
- Driver population factor f_p = 1.00;
- Proportion of AADT in peak hour (*K*-factor): 0.08; and
- Directional distribution (*D*-factor): 60/40.

Comments

In this problem, the service flow rate, service volume, and daily service volume for each LOS will be computed. These values could then be compared with any existing or forecast demand volumes to determine the LOS.

Step 1: Input Data

All input data were specified.

Step 2: Compute FFS

The FFS has been field-measured as 70 mi/h.

Step 3: Select FFS Curve

The curve for FFS = 70 mi/h will be used.

Step 4: Compute Service Flow Rates, *SF*

For a 70-mi/h basic freeway segment, maximum service flow rates, *MSF*, can be selected from Exhibit 11-17. These are the maximum service flow rates that can be sustained while a given LOS is maintained. They are stated as flow rates in passenger cars per hour per lane for equivalent base conditions. The values are

- MSF_A = 770 pc/h/ln,
- MSF_B = 1,250 pc/h/ln,
- MSF_C = 1,690 pc/h/ln,
- MSF_D = 2,080 pc/h/ln, and
- MSF_E = 2,400 pc/h/ln.

Service flow rates, *SF*, are estimated by using Equation 11-9:

$$SF_i = MSF_i \times N \times f_{HV} \times f_p$$

where the maximum service flow rates are as cited, $N = 4$ lanes in each direction, and the driver population factor f_p is 1.00. The heavy-vehicle adjustment factor must be determined for 8% trucks and 1% RVs in rolling terrain. From Exhibit 11-10, for rolling terrain, $E_T = 2.5$ and $E_R = 2.0$. Then

$$f_{HV} = \frac{1}{1 + 0.08(2.5 - 1) + 0.01(2.0 - 1)} = 0.885$$

Service flow rates may now be computed:

$$SF_A = 770 \times 4 \times 0.885 \times 1.00 = 2{,}726 \text{ veh/h}$$

$$SF_B = 1{,}250 \times 4 \times 0.885 \times 1.00 = 4{,}425 \text{ veh/h}$$

$$SF_C = 1{,}690 \times 4 \times 0.885 \times 1.00 = 5{,}983 \text{ veh/h}$$

$$SF_D = 2{,}080 \times 4 \times 0.885 \times 1.00 = 7{,}363 \text{ veh/h}$$

$$SF_E = 2{,}400 \times 4 \times 0.885 \times 1.00 = 8{,}496 \text{ veh/h}$$

Service flow rates are the maximum rates of flow that may exist in the worst 15-min period of the peak hour while the stated LOS is maintained.

Step 5: Compute Service Volumes, *SV*

Equation 11-10 is used to convert service flow rates to service volumes. The conversion multiplies the service flow rates by the PHF to produce maximum hourly volumes that can be accommodated while the given LOS is maintained during the worst 15 min of the hour.

$$SV_i = SF_i \times PHF$$

$$SV_A = 2{,}726 \times 0.87 = 2{,}372 \text{ veh/h}$$

$$SV_B = 4{,}425 \times 0.87 = 3{,}850 \text{ veh/h}$$

$$SV_C = 5{,}983 \times 0.87 = 5{,}205 \text{ veh/h}$$

$$SV_D = 7{,}363 \times 0.87 = 6{,}406 \text{ veh/h}$$

$$SV_E = 8{,}496 \times 0.87 = 7{,}392 \text{ veh/h}$$

Step 6: Compute Daily Service Volumes, *DSV*

Equation 11-11 is used to convert service volumes to daily service volumes. Daily service volumes are the maximum AADTs that can be accommodated while the given LOS is maintained during the worst 15 min of the peak hour in the peak direction of flow.

$$DSV_i = \frac{SV_i}{K \times D}$$

$$DSV_A = \frac{2{,}372}{0.08 \times 0.60} = 49{,}417 \text{ veh/day}$$

$$DSV_B = \frac{3{,}850}{0.08 \times 0.60} = 80{,}208 \text{ veh/day}$$

$$DSV_C = \frac{5{,}205}{0.08 \times 0.60} = 108{,}438 \text{ veh/day}$$

$$DSV_D = \frac{6{,}406}{0.08 \times 0.60} = 133{,}458 \text{ veh/day}$$

$$DSV_E = \frac{7{,}392}{0.08 \times 0.60} = 154{,}000 \text{ veh/day}$$

Discussion

These results can be conveniently shown in the form of a table, as illustrated in Exhibit 11-22. Given the approximate nature of these computations and the default values used, it is appropriate to round the DSV values to the nearest 100 veh/day, and SF and SV values to the nearest 10 veh/h.

Exhibit 11-22
Service Flow Rates, Service Volumes, and Daily Service Volumes for Example Problem 6

LOS	*SF* (veh/h)	*SV* (veh/h)	*DSV* (veh/day)
A	2,730	2,370	49,400
B	4,430	3,850	80,200
C	5,980	5,210	108,400
D	7,360	6,410	133,500
E	8,500	7,390	154,000

Exhibit 11-22, of course, applies only to the basic freeway segment as described. Should any of the prevailing conditions change, the values in the exhibit would also change. However, for a given segment, forecast demand volumes, whether given as flow rates, hourly volumes, or AADTs, could be compared with the criteria in Exhibit 11-22 to determine the likely LOS immediately. For example, if the 10-year forecast AADT for this segment is 125,000 veh/day, the expected LOS would be D.

5. REFERENCES

Many of these references can be found in the Technical Reference Library in Volume 4.

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APPENDIX A: COMPOSITE GRADES

In a basic freeway segment analysis, an overall average grade can be substituted for a series of grades if no single portion of the grade is steeper than 4% or the total length of the grade is less than 4,000 ft. For grades outside these limits (i.e., a portion of the grade is greater than 4% *and* the total length of the grade is greater than or equal to 4,000 ft), the composite grade procedure presented in this appendix is recommended. The composite grade procedure is used to determine an equivalent grade that will result in the same final speed of trucks as would the series of grades making up the composite.

The composite grade procedure should be used for a series of grades that are ≥4,000 ft in length and that have a portion of the grade steeper than 4%.

The procedure finds the equivalent single grade that results in the same final truck speed as the series of grades would.

The acceleration and deceleration curves presented here are for vehicles with an average weight-to-horsepower ratio of 200 lb/hp, heavier than typical trucks found on freeways, which range between 125 lb/hp and 150 lb/hp. This is done in recognition of the fact that heavier trucks will have more of an impact on the traffic stream than lighter trucks.

Exhibit 11-A1 shows typical acceleration (*dashed lines*) and deceleration (*solid lines*) performance for a truck with a ratio of 200 lb/hp. The curves are conservative in that they assume a maximum truck speed of 55 mi/h for trucks entering a grade and 60 mi/h for trucks accelerating on a grade.

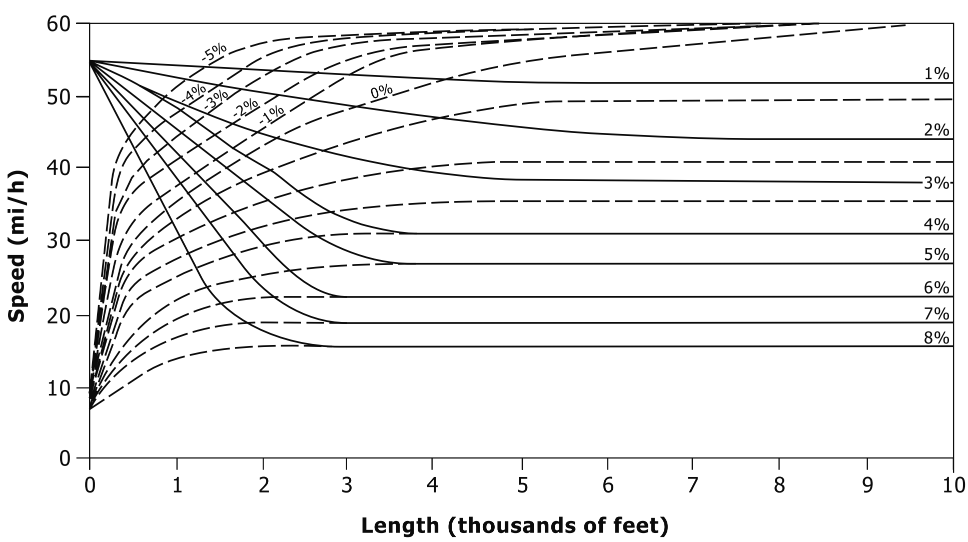


Exhibit 11-A1
Performance Curves for 200-lb/hp Truck

LIVE GRAPH
Click here to view

EXAMPLE PROBLEM

An example is provided to illustrate the process involved in determining an equivalent grade for a composite grade on a freeway. The example has two segments, but the procedure is valid for any number of segments. The composite grade is

- Upgrade of 2% for 5,000 ft, followed by
- Upgrade of 6% for 5,000 ft.

This grade should not be analyzed with an average grade approach, because one portion of the grade is steeper than 4% and the total length of the grade is in

excess of 4,000 ft. As a comparison, application of the average grade approach in this case would yield the following:

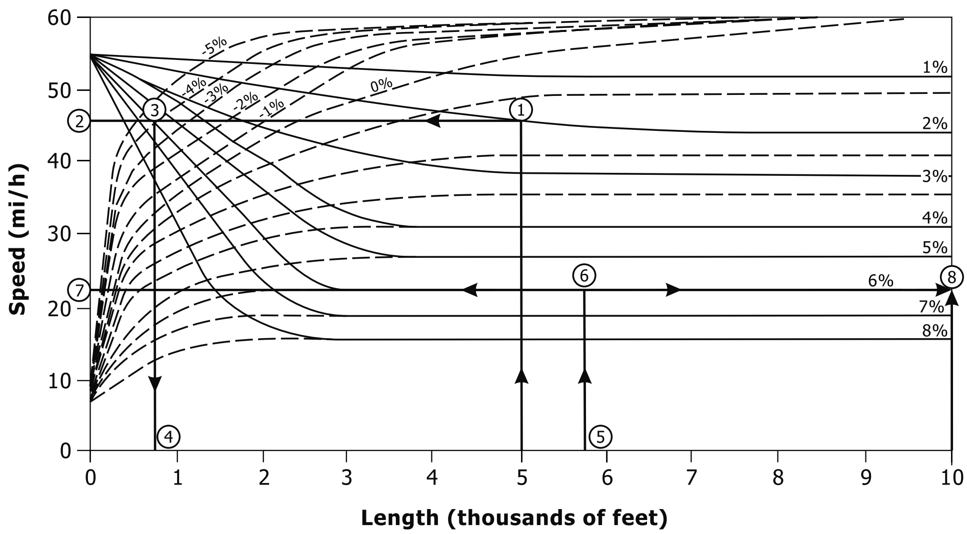
- Total rise along composite: (5,000 × 0.02) + (5,000 × 0.06) = 400 ft.
- Average grade: 400/10,000 = 0.04, or 4%.

With the average grade approach, the composite would be analyzed as if it were a single upgrade of 4%, 10,000 ft (1.89 mi) long.

Exhibit 11-A2 illustrates the recommended solution.

Exhibit 11-A2
Solution Using Composite Grade Procedure

LIVE GRAPH
Click here to view



The flat portions of the upgrade curves indicate the truck crawl speed for that grade.

A vertical line is drawn at 5,000 ft to the intersection with the curve for the +2% grade (Point 1). A horizontal line is drawn from the intersection point to the *y*-axis (Point 2). This procedure indicates that after 5,000 ft of +2% upgrade, trucks will be operating at a speed of approximately 46 mi/h.

This speed is also the speed at which trucks enter the +6% segment of the composite grade. The intersection of the 46-mi/h horizontal line with the curve for the +6% grade (Point 3) is found. A vertical line is dropped from this point to the *x*-axis (Point 4). This procedure indicates that trucks enter the +6% segment of the composite as if they had already been on the +6% grade for approximately 800 ft. Trucks will travel another 5,000 ft along the +6% grade, starting from Point 4. A vertical line is drawn at a distance of 800 + 5,000 = 5,800 ft (Point 5) to the intersection with the curve for the +6% grade (Point 6). A horizontal line drawn from this point to the *y*-axis (Point 7) indicates that the speed of trucks at the end of the two-segment composite grade will be approximately 23 mi/h.

The solution point is found as the intersection of a vertical line drawn at 10,000 ft (the total length of the composite grade) and a horizontal line drawn at 23 mi/h. The solution is read as the percent grade on which the solution point lies (Point 8). In this case, the point lies exactly on the curve for the 6% grade. Interpolations between curves are permissible.

In this case, the grade that is equivalent to the composite grade is a single grade of 6%, 10,000 ft (1.89 mi) long. This grade is 2% higher than the 4% average grade. The appropriate equivalent grade is the same percentage as the second

segment of the composite grade because trucks have already reached crawl speed. Once trucks hit crawl speed, it does not matter how far from the beginning of the grade they are; their speed will remain constant.

PROCEDURAL STEPS

The general steps taken in solving for a composite-grade equivalent are summarized as follows:

1. Enter Exhibit 11-A1 with the length of the first segment of the composite grade.
2. Find the truck speed at the end of the first segment of the grade.
3. Find the length along the second segment of the grade that results in the same speed as that found in Step 2.
4. Add the length of the Segment 2 grade to the length determined in Step 3.
5. Repeat Steps 2 through 4 for each subsequent grade segment.
6. Find the intersection of a vertical line drawn at the total length of the composite grade and a horizontal line drawn at the final speed of trucks at the end of the composite grade.
7. Determine the percent of grade for the solution point of Step 6.

DISCUSSION

In the analysis of composite grades, the point of interest is not always at the end of the grade. It is important to identify the point at which the speed of trucks is the lowest because this is where trucks will have the maximum impact on operating conditions. This point may be an intermediate point. If a +3% grade of 1,000 ft is followed by a +4% grade of 2,000 ft, then by a +2% grade of 1,500 ft, the speed of trucks will be slowest at the end of the +4% grade segment. Thus, a composite grade solution would be sought for the first two segments of the grade, with a total grade length of 1,000 + 2,000 = 3,000 ft.

The composite grade procedure is not applicable in all cases, especially if the first segment is a downgrade and the segment length is long or if the segments are too short. In the use of performance curves, cases that cannot be solved with this procedure will become apparent to the analyst because the line will not intersect or the points will fall outside the limits of the curves. In such cases, field measurements of speeds should be used as inputs to the selection of appropriate truck equivalency values.

CHAPTER 12
FREEWAY WEAVING SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Weaving is generally defined as the crossing of two or more traffic streams traveling in the same direction along a significant length of highway without the aid of traffic control devices (except for guide signs). Thus, weaving segments are formed when merge segments are closely followed by diverge segments. “Closely” implies that there is not sufficient distance between the merge and diverge segments for them to operate independently.

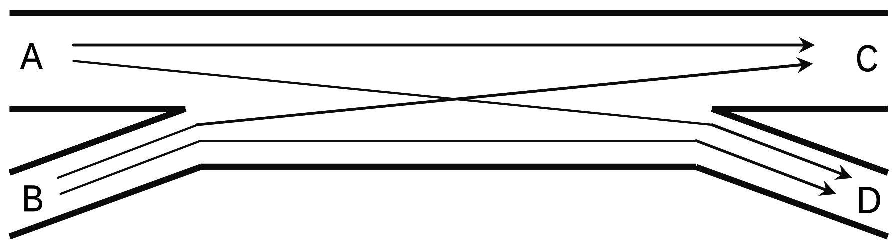
Three geometric characteristics affect a weaving segment’s operating characteristics: length, width, and configuration. All have an impact on the critical lane-changing activity, which is the unique operating feature of a weaving segment. **Chapter 12, Freeway Weaving Segments,** provides a methodology for analyzing the operation of weaving segments based on these characteristics as well as a segment’s free-flow speed (FFS) and the demand flow rates for each movement within a weaving segment (e.g., ramp to freeway or ramp to ramp). This chapter describes how the methodology can be applied to planning, operations, and design applications and provides examples of these applications.

2. WEAVING SEGMENT CHARACTERISTICS

OVERVIEW

Exhibit 12-1 illustrates a freeway weaving segment. On entry and exit roadways, or *legs*, vehicles traveling from Leg A to Leg D must cross the path of vehicles traveling from Leg B to Leg C. Flows A–D and B–C are, therefore, referred to as *weaving movements*. Flows A–C and B–D may also exist, but as they are not required to cross the path of any other flow, they are referred to as *nonweaving movements*.

Exhibit 12-1
Formation of a Weaving Segment



Traffic in a weaving segment experiences more lane-changing turbulence than is normally present on basic freeway segments.

Weaving segments require intense lane-changing maneuvers as drivers must access lanes appropriate to their desired exit leg. Therefore, traffic in a weaving segment is subject to lane-changing turbulence in excess of that normally present on basic freeway segments. This additional turbulence presents operational problems and design requirements, which are addressed by this chapter's methodology.

A weaving segment's geometry affects its operating characteristics.

Three geometric characteristics affect a weaving segment's operating characteristics:

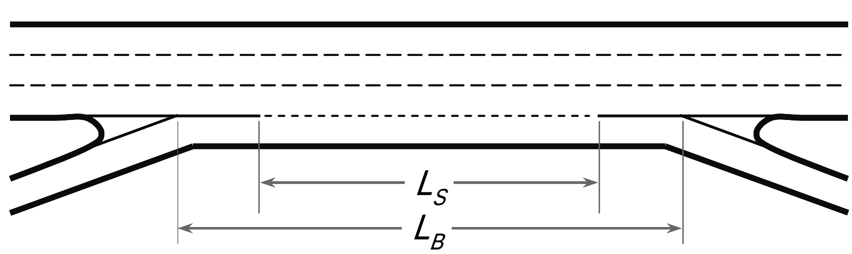
- Length,
- Width, and
- Configuration.

Length is the distance between the merge and diverge that form the weaving segment. *Width* refers to the number of lanes within the weaving segment. *Configuration* is defined by the way entry and exit lanes are aligned. All have an impact on the critical lane-changing activity, which is the unique operating feature of a weaving segment.

LENGTH OF A WEAVING SEGMENT

The two measures of weaving segment length that are relevant to this chapter's methodology are illustrated in Exhibit 12-2.

Exhibit 12-2
Measuring the Length of a Weaving Segment



The lengths illustrated are defined as follows:

L_S = short length, the distance in feet between the end points of any barrier markings (solid white lines) that prohibit or discourage lane changing.

L_B = base length, the distance in feet between points in the respective gore areas where the left edge of the ramp-traveled way and the right edge of the freeway-traveled way meet.

Neither of these definitions is the same as those used in previous editions of the *Highway Capacity Manual* (HCM). The definitions used throughout the HCM2000 were historically tied to the specifics of the design of loop ramps in a cloverleaf interchange at a time when most weaving segments were part of such interchanges. Modern weaving segments occur in a wide range of situations and designs, and a more general definition of length is appropriate.

The weaving segment length used in the methodology is defined by the distance between barrier markings. Where no markings exist, the length is defined by the distance between where the left edge of the ramp-traveled way and the right edge of the freeway-traveled way meet.

This methodology includes several equations that include the length of the weaving segment. In all cases, these equations use the short length L_S. This is not to suggest that lane changing in a weaving segment is restricted to this length. Some lane changing takes place over solid white lines and even painted gore areas. Nevertheless, research has shown that the short length is a better predictor of operating characteristics within the weaving segment than either the base length or the length as defined in HCM2000 and previous editions.

For weaving segments in which no solid white lines are used, the two lengths illustrated in Exhibit 12-2 are the same, that is, $L_S = L_B$. In dealing with future designs in which the details of markings are unknown, a default value should be based on the general marking policy of the operating agency. At the time this methodology was developed, where solid white lines were provided, L_S was equal to $0.77 \times L_B$ on average for the available data.

The estimated speeds and densities, however, apply over the base length L_B. Some evidence also indicates that these speeds and densities may apply to the 500 ft of freeway upstream of the merge and downstream of the diverge because of presegregation of movements in each case.

Under constant demand conditions, making a weaving segment longer increases its capacity and improves its operation.

The weaving segment length strongly influences lane-changing intensity. For any given demand situation, longer segments allow weaving motorists more time and space to execute their lane changes. This reduces the density of lane changing and, therefore, turbulence. Lengthening a weaving segment both increases its capacity and improves its operation (assuming a constant demand).

WIDTH OF A WEAVING SEGMENT

The number of continuous lanes between gore areas within a weaving segment defines its width.

The width of a weaving segment is measured as the number of continuous lanes within the segment, that is, the number of continuous lanes between the entry and exit gore areas. Acceleration or deceleration lanes that extend partially into the weaving segment are not included in this count.

While additional lanes provide more space for both weaving and nonweaving vehicles, they encourage additional optional lane-changing activity. Thus, while reducing overall densities, additional lanes can increase lane-changing activity and intensity. In most cases, however, the number of lanes in

the weaving segment is controlled by the number of lanes on the entry and exit legs and the intended configuration.

CONFIGURATION OF A WEAVING SEGMENT

Configuration of a weaving segment refers to the way that entry and exit lanes are linked. The configuration determines how many lane changes a weaving driver must make to complete the weaving maneuver successfully. The following sections use a great deal of terminology to describe configurations; this terminology should be clearly understood.

One-Sided and Two-Sided Weaving Segments

One-sided weaving segments require no more than two lane changes to complete a weaving maneuver.

Most weaving segments are one-sided. In general, this means that the ramps defining the entry to and exit from the weaving segment are on the same side of the freeway—either both on the right (most common) or both on the left. The methodology of this chapter was developed for one-sided weaving segments; however, guidelines are given for applying the methodology to two-sided weaving segments.

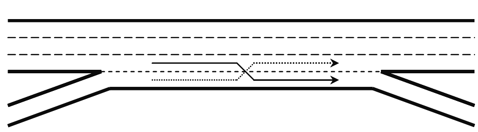
Two-sided weaving segments require three or more lane changes to complete a weaving maneuver or have a single-lane on-ramp closely followed by a single-lane off-ramp on the opposite side of the freeway.

One- and two-sided weaving segments are defined as follows:

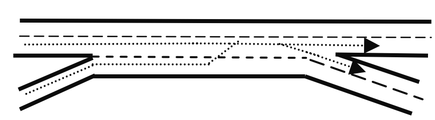
- A *one-sided weaving segment* is one in which no weaving maneuvers require more than two lane changes to be completed successfully.
- A *two-sided weaving segment* is one in which at least one weaving maneuver requires three or more lane changes to be completed successfully or in which a single-lane on-ramp is closely followed by a single-lane off-ramp on the opposite side of the freeway.

Exhibit 12-3 illustrates two examples of one-sided weaving segments.

Exhibit 12-3
One-Sided Weaving Segments Illustrated



(a) One-Sided Ramp Weave

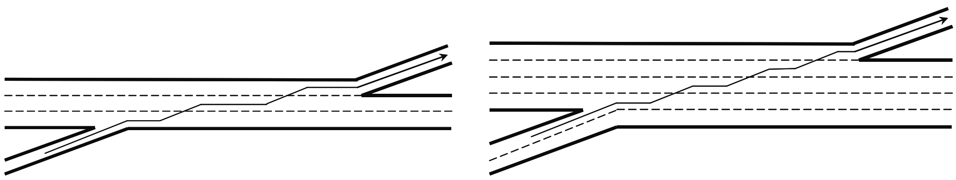


(b) One-Sided Major Weave

Exhibit 12-3(a) shows a typical one-sided weaving segment formed by a one-lane, right-side on-ramp followed closely by a one-lane, right-side off-ramp. The two are connected by a continuous freeway auxiliary lane. Every weaving vehicle must make one lane change as illustrated, and the lane-changing turbulence caused is clearly focused on the right side of the freeway. Exhibit 12-3(b) shows another one-sided weaving segment in which the off-ramp has two lanes. One weaving movement (ramp to freeway) requires one lane change. The other (freeway to ramp) can be made without making a lane change. Again, lane-changing turbulence is focused on the right side of the freeway.

Exhibit 12-4 contains two examples of two-sided weaving segments.

Exhibit 12-4
Two-Sided Weaving Segments Illustrated



(a) Two-Sided Weaving Section with Single-Lane Ramps

(b) Two-Sided Weaving Section with Three Lane Changes

Exhibit 12-4(a) is the most common form of a two-sided weave. A one-lane, right-side on-ramp is closely followed by a one-lane, left-side off-ramp (or vice versa). Although the ramp-to-ramp weaving movement requires only two lane changes, this movement is still classified as a two-sided weave because the geometry of the through movement on the freeway technically qualifies as a weaving flow.

Exhibit 12-4(b) is a less typical case in which one of the ramps has multiple lanes. Because the ramp-to-ramp weaving movement must execute three lane changes, it is also classified as a two-sided weaving segment.

Ramp-Weave Versus Major Weave Segments

Exhibit 12-3 can also be used to illustrate the difference between a ramp-weaving segment and a major weaving segment. Exhibit 12-3(a) shows a typical ramp-weaving segment, formed by a one-lane on-ramp closely followed by a one-lane off-ramp, connected by a continuous freeway auxiliary lane. The unique feature of the ramp-weave configuration is that all weaving drivers must execute a lane change across the lane line separating the freeway auxiliary lane from the right lane of the freeway mainline.

It is important to note that the case of a one-lane on-ramp closely followed by a one-lane off-ramp (on the same side of the freeway), but not connected by a continuous freeway auxiliary lane, is not considered to be a weaving configuration. Such cases are treated as isolated merge and diverge segments by using the methodology described in Chapter 13. The distance between the on-ramp and the off-ramp is not a factor in this determination.

One-sided configurations without a continuous auxiliary lane connecting an on-ramp to a closely following off-ramp are treated as isolated ramp junctions (Chapter 13) and not as weaving segments.

Exhibit 12-3(b) shows a typical major weaving segment. A major weaving segment is formed when three or more entry or exit legs have multiple lanes.

Numerical Measures of Configuration

Three numerical descriptors of a weaving segment characterize its configuration:

LC_{RF} = minimum number of lane changes that a ramp-to-freeway weaving vehicle must make to complete the ramp-to-freeway movement successfully.

LC_{FR} = minimum number of lane changes that a freeway-to-ramp weaving vehicle must make to complete the freeway-to-ramp movement successfully.

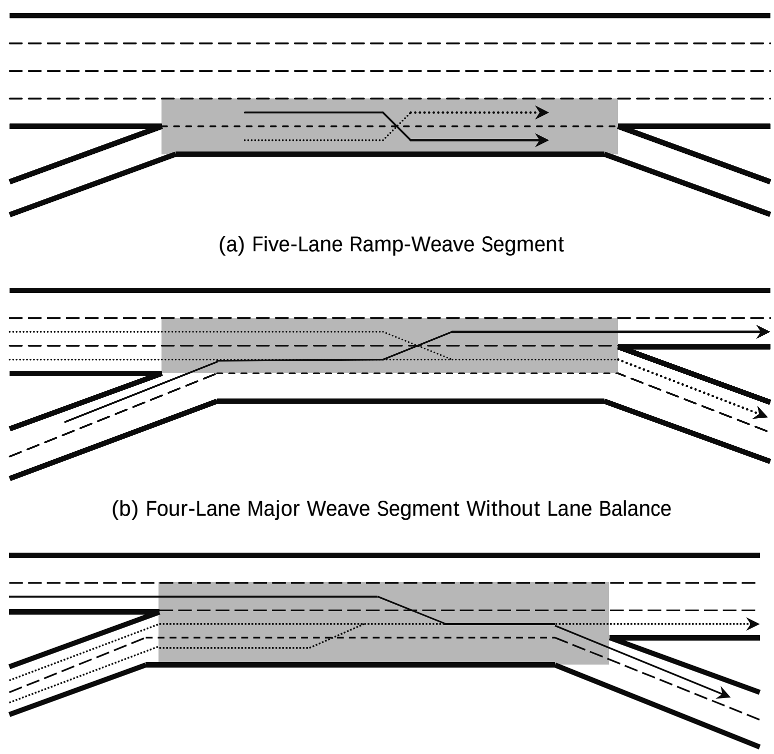
"Minimum number of lane changes" assumes vehicles position themselves when entering and exiting to make the least number of lane changes possible.

N_{WL} = number of lanes from which a weaving maneuver may be completed with one lane change or no lane changes.

These definitions apply directly to one-sided weaving segments in which the ramp-to-freeway and freeway-to-ramp movements are the weaving movements. Different definitions apply to two-sided weaving segments. Exhibit 12-5 illustrates how these values are determined for one-sided weaving segments.

The values of LC_{RF} and LC_{FR} are found by assuming that every weaving vehicle enters the segment in the lane closest to its desired exit leg and leaves the segment in the lane closest to its entry leg.

Exhibit 12-5
Configuration Parameters Illustrated



(a) Five-Lane Ramp-Weave Segment

(b) Four-Lane Major Weave Segment Without Lane Balance

(c) Four-Lane Major Weave Segment With Lane Balance

Exhibit 12-5(a) is a five-lane ramp-weave configuration. If a weaving driver wishes to exit on the off-ramp and enters the segment on the rightmost freeway lane (the lane closest to the off-ramp), the driver must make a single lane change to enter the freeway auxiliary lane and leave via the off-ramp. Thus, for this case, $LC_{FR} = 1$. A weaving driver entering the freeway via the on-ramp has no choice but to enter on the freeway auxiliary lane. The driver must then make a single lane change from the freeway auxiliary lane to the rightmost lane of the freeway (the lane closest to the entry leg). Thus, $LC_{RF} = 1$ as well.

Lane balance within a weaving segment provides operational flexibility.

Exhibit 12-5(b) and Exhibit 12-5(c) are both major weaving configurations consisting of four lanes. They differ only in the configuration of their entry and exit gore areas. One has lane balance, while the other does not. Lane balance exists when the number of lanes leaving a diverge segment is one more than the number of lanes entering it.

Exhibit 12-5(b) is not typical. It is used here only to demonstrate the concept of lane balance in a major weaving segment. Five lanes approach the entry to the segment and four lanes leave it; four lanes approach the exit from the segment and four lanes leave it. Because of this configuration, vehicles approaching the exit gore must already be in an appropriate lane for their intended exit leg.

In Exhibit 12-5(b), the ramp-to-freeway weaving movement (right to left) requires at least one lane change. A vehicle can enter the segment on the leftmost ramp lane (the lane closest to the desired exit) and make a single lane change to exit on the rightmost lane of the continuing freeway. LC_{RF} for this case is 1. The freeway-to-ramp weaving movement can be made without any lane changes. A vehicle can enter on the rightmost lane of the freeway and leave on the leftmost lane of the ramp without executing a lane change. For this case, $LC_{FR} = 0$.

The exit junction in Exhibit 12-5(c) has lane balance: four lanes approach the exit from the segment and five lanes leave it. This is a desirable feature that provides some operational flexibility. One lane—in this case, the second lane from the right—splits at the exit. A vehicle approaching in this lane can take either exit leg without making a lane change. This is a useful configuration in cases in which the split of exiting traffic varies over a typical day. The capacity provided by the splitting lane can be used as needed by vehicles destined for either exit leg.

In Exhibit 12-5(c), the ramp-to-freeway movement can be made without a lane change, while the freeway-to-ramp movement requires a single lane change. For this case, $LC_{RF} = 0$ and $LC_{FR} = 1$.

In Exhibit 12-5(a), there are only two lanes from which a weaving movement may be made with no more than one lane change. Weaving vehicles may enter the segment in the freeway auxiliary lane (ramp-to-freeway vehicles) and in the rightmost freeway lane (freeway-to-ramp vehicles) and may execute a weaving maneuver with a single lane change. Although freeway-to-ramp vehicles may enter the segment on the outer freeway lanes, they would have to make more than one lane change to access the off-ramp. Thus, for this case, $N_{WL} = 2$.

In Exhibit 12-5(b), weaving vehicles entering the segment in the leftmost lane of the on-ramp or the rightmost lane of the freeway are forced to merge into a single lane. From this lane, the freeway-to-ramp movement can be made with no lane changes, while the ramp-to-freeway movement requires one lane change. Because the movements have merged into a single lane, this counts as one lane from which weaving movements can be made with one or fewer lane changes. Freeway-to-ramp vehicles, however, may also enter the segment on the center lane of the freeway and make a single lane change (as shown) to execute their desired maneuver. Thus, for this case, N_{WL} is once again 2.

Lane balance creates more flexibility in Exhibit 12-5(c). Ramp-to-freeway vehicles may enter on either of the two lanes of the on-ramp and complete a weaving maneuver with either one or no lane changes. Freeway-to-ramp vehicles may enter on the rightmost freeway lane and also weave with a single lane change. In this case, $N_{WL} = 3$.

In all one-sided weaving segments, the number of lanes from which weaving maneuvers may be made with one or no lane changes is either two or three. No other values are possible. Segments with $N_{WL} = 3$ generally exist in major weaving segments with lane balance at the exit gore.

Special Case: Two-Sided Weaving Segments

Only the ramp-to-ramp movement is considered to be a weaving flow in a two-sided weaving segment.

The parameters defining the impact of configuration apply only to one-sided weaving segments. In a two-sided weaving segment, neither the ramp-to-freeway nor the freeway-to-ramp movements weave. While the through freeway movement in a two-sided weaving segment might be functionally thought of as weaving, it is the dominant movement in the segment and does not behave as a weaving movement. Thus, in two-sided weaving segments, only the ramp-to-ramp movement is considered to be a weaving flow. This introduces two specific changes to the methodology:

1. Instead of LC_{RF} and LC_{FR} being needed to characterize weaving behavior, a value of LC_{RR} (the minimum number of lane changes that must be made by a ramp-to-ramp vehicle) is needed. In Exhibit 12-4(a), $LC_{RR} = 2$, while in Exhibit 12-4(b), $LC_{RR} = 3$.
2. In all cases of two-sided weaving, the value of N_{WL} is set to 0 by definition.

With these two modifications, the methodology outlined for one-sided weaving segments may be applied to two-sided weaving segments as well.

3. METHODOLOGY

The methodology presented in this chapter was developed as part of National Cooperative Highway Research Program (NCHRP) Project 3-75, *Analysis of Freeway Weaving Sections* (*1*). Elements of this methodology have also been adapted from earlier studies and earlier editions of this manual (*2*–*9*).

LIMITATIONS OF THE METHODOLOGY

The methodology of this chapter does not specifically address the following subjects (without modifications by the analyst):

- Special lanes, such as high-occupancy vehicle lanes, within the weaving segment;
- Ramp metering on entrance ramps forming part of the weaving segment;
- Specific operating conditions when oversaturated conditions exist;
- Effects of speed limit enforcement practices on weaving segment operations;
- Effects of intelligent transportation system technologies on weaving segment operations;
- Weaving segments on arterials or other urban streets, including one-way frontage roads;
- Effects of downstream congestion or upstream demand starvation on the analysis segment; or
- Multiple weaving segments.

The last subject has been included in previous versions of this manual. Multiple weaving segments must now be divided into appropriate merge, diverge, and simple weaving segments for analysis.

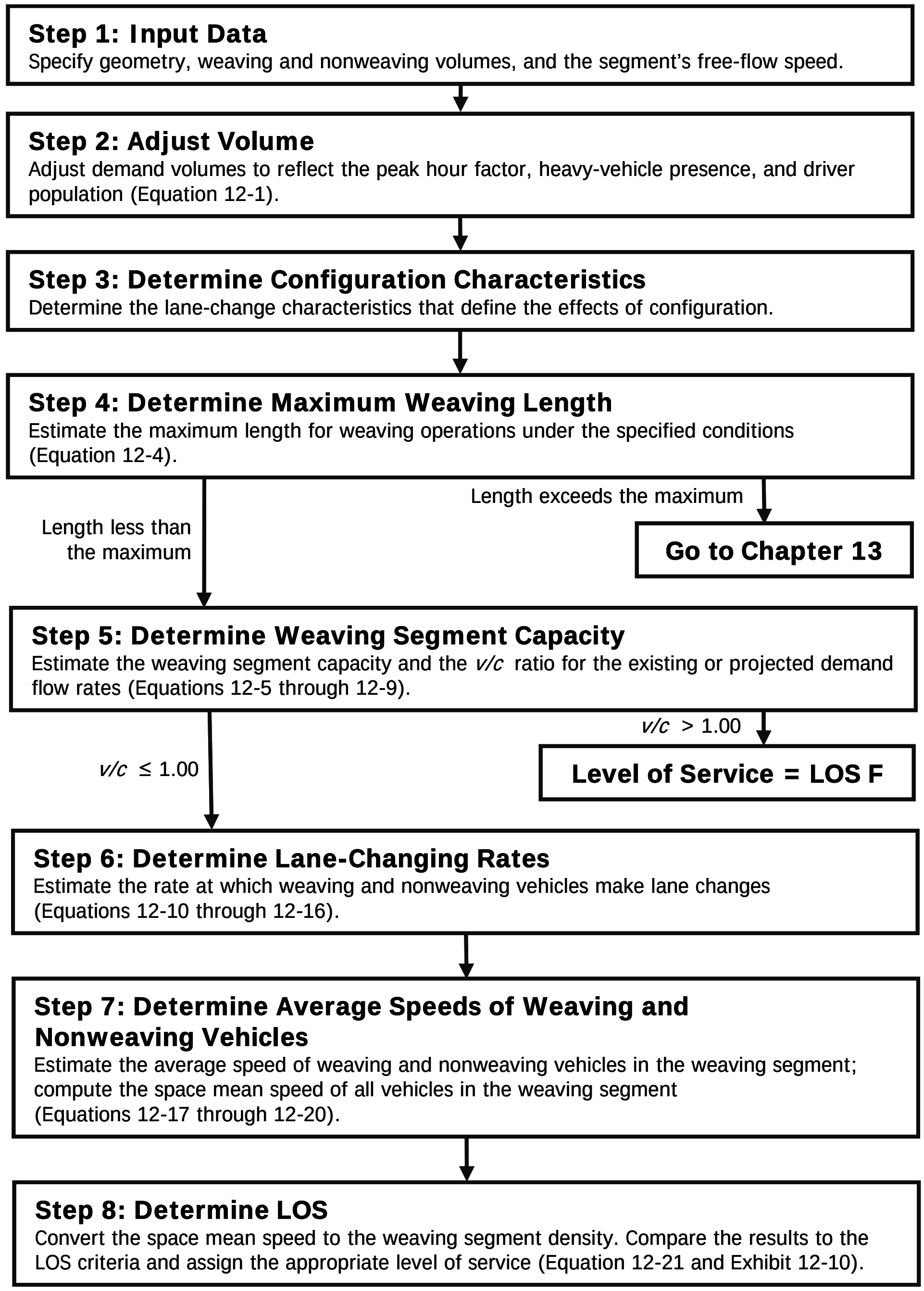
Multiple weaving segments must be divided into merge, diverge, and simple weaving segments for analysis.

OVERVIEW OF THE METHODOLOGY

Exhibit 12-6 is a flowchart illustrating the basic steps that define the methodology for analyzing freeway weaving segments. The methodology uses several types of predictive algorithms, all of which are based on a mix of theoretical and regression models. These models include the following:

- Models that predict the total rate of lane changing taking place in the weaving segment. This is a direct measure of turbulence in the traffic stream caused by the presence of weaving movements.
- Models to predict the average speed of weaving and nonweaving vehicles in a weaving segment under stable operating conditions, that is, not operating at Level of Service (LOS) F.
- Models to predict the capacity of a weaving segment under both ideal and prevailing conditions.
- A model to estimate the maximum length over which weaving operations can be said to exist.

Exhibit 12-6
Weaving Methodology Flowchart



If the potential weaving segment is longer than the value given by Equation 12-4, it is treated as isolated merging and diverging ramp junctions by using the procedures of Chapter 13.

LOS F exists in a weaving segment when demand exceeds capacity.

PARAMETERS DESCRIBING A WEAVING SEGMENT

Several parameters describing weaving segments have already been introduced and defined. Exhibit 12-7 illustrates all variables that must be specified as input variables and defines those that will be used within or as outputs of the methodology. Some of these apply only to one-sided weaving segments. Exhibit 12-8 lists those variables that are different when applied to two-sided weaving segments.

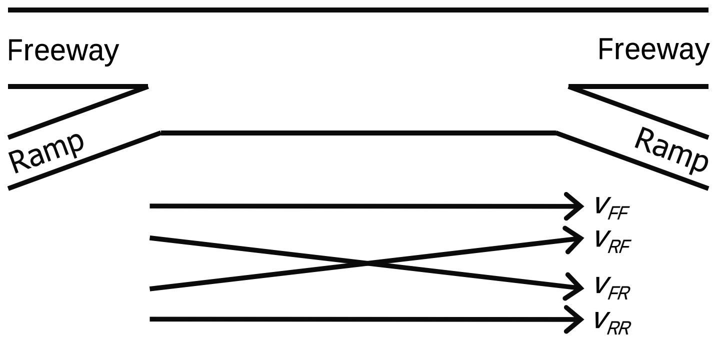


Exhibit 12-7
Weaving Variables for One-Sided Weaving Segments

v_{FF} = freeway-to-freeway demand flow rate in the weaving segment in passenger cars per hour (pc/h);

v_{RF} = ramp-to-freeway demand flow rate in the weaving segment (pc/h);

v_{FR} = freeway-to-ramp demand flow rate in the weaving segment (pc/h);

v_{RR} = ramp-to-ramp demand flow rate in the weaving segment (pc/h);

v_W = weaving demand flow rate in the weaving segment (pc/h), $v_W = v_{RF} + v_{FR}$;

v_{NW} = nonweaving demand flow rate in the weaving segment (pc/h), $v_{NW} = v_{FF} + v_{RR}$;

v = total demand flow rate in the weaving segment (pc/h), $v = v_W + v_{NW}$;

VR = volume ratio, v_W/v;

N = number of lanes within the weaving section;

N_{WL} = number of lanes from which a weaving maneuver may be made with one or no lane changes (see Exhibit 12-5);

S_W = average speed of weaving vehicles within the weaving segment (mi/h);

S_{NW} = average speed of nonweaving vehicles within the weaving segment (mi/h);

S = average speed of all vehicles within the weaving segment (mi/h);

FFS = free-flow speed of the weaving segment (mi/h);

D = average density of all vehicles within the weaving segment in passenger cars per mile per lane (pc/mi/ln);

W = weaving intensity factor;

L_S = length of the weaving segment (ft), based on the short length definition of Exhibit 12-2;

LC_{RF} = minimum number of lane changes that must be made by a single weaving vehicle moving from the on-ramp to the freeway (see Exhibit 12-5);

LC_{FR} = minimum number of lane changes that must be made by a single weaving vehicle moving from the freeway to the off-ramp;

LC_{MIN} = minimum rate of lane changing that must exist for all weaving vehicles to complete their weaving maneuvers successfully, in lane changes per hour (lc/h), $LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$;

LC_W = total rate of lane changing by weaving vehicles within the weaving segment (lc/h);

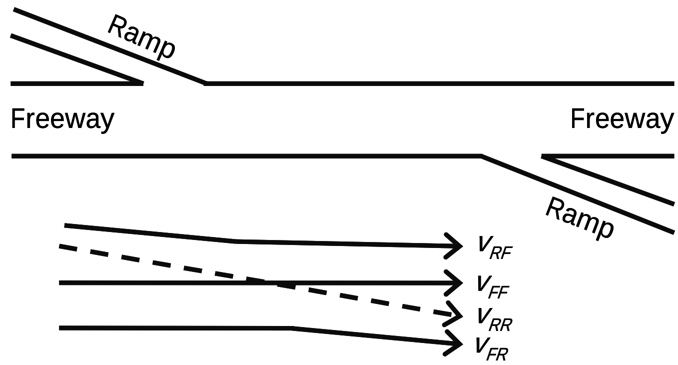
LC_{NW} = total rate of lane changing by nonweaving vehicles within the weaving segment (lc/h);

LC_{ALL} = total rate of lane changing of all vehicles within the weaving segment (lc/h), $LC_{ALL} = LC_W + LC_{NW}$;

ID = interchange density, the number of interchanges within ±3 mi of the center of the subject weaving segment divided by 6, in interchanges per mile (int/mi); and

I_{LC} = lane-changing intensity, LC_{ALL}/L_S, in lane changes per foot (lc/ft).

Exhibit 12-8
Weaving Variables for a Two-Sided Weaving Segment



The through freeway movement is not considered to be weaving in a two-sided weaving segment.

All variables are defined as in Exhibit 12-7, except for the following variables relating to flow designations and lane-changing variables:

v_W = total weaving demand flow rate within the weaving segment (pc/h), $v_W = v_{RR}$;

v_{NW} = total nonweaving demand flow rate within the weaving segment (pc/h), $v_{NW} = v_{FR} + v_{RF} + v_{FF}$;

LC_{RR} = minimum number of lane changes that must be made by one ramp-to-ramp vehicle to complete a weaving maneuver; and

LC_{MIN} = minimum rate of lane changing that must exist for all weaving vehicles to complete their weaving maneuvers successfully (lc/h), $LC_{MIN} = LC_{RR} \times v_{RR}$.

The principal difference between one-sided and two-sided weaving segments is the relative positioning of the movements within the segment. In a two-sided weaving segment, the ramp-to-freeway and freeway-to-ramp vehicles do not weave. In a one-sided segment, they execute the weaving movements. In a two-sided weaving segment, the ramp-to-ramp vehicles must cross the path of freeway-to-freeway vehicles. Both could be taken to be weaving movements. In reality, the through freeway movement is not weaving in that vehicles do not need to change lanes and generally do not shift lane position in response to a desired exit leg.

Thus, in two-sided weaving segments, only the ramp-to-ramp flow is considered to be weaving. The lane-changing parameters reflect this change in the way weaving flows are viewed. Thus, the minimum rate of lane changing that weaving vehicles must maintain to complete all desired weaving maneuvers successfully is also related only to the ramp-to-ramp movement.

The methodology uses demand flow rates for the peak 15 min in passenger cars per hour.

The definitions for flow all refer to *demand flow rate*. This means that for existing cases, the demand should be based on *arrival flows*. For future cases, forecasting techniques will generally produce a *demand volume* or *demand flow rate*. All of the methodology's algorithms use demand expressed as flow rates in the peak 15 min of the design (or analysis) hour, in equivalent passenger car units.

COMPUTATIONAL PROCEDURES

Each of the major procedural steps noted in Exhibit 12-6 is discussed in detail in the sections that follow.

Step 1: Input Data

The methodology for weaving segments is structured for operational analysis usage, that is, given a known or specified geometric design and traffic demand characteristics, the methodology is used to estimate the LOS that is expected to exist.

Design and preliminary engineering are generally conducted in terms of comparative analyses of various design proposals. This is a good approach, given that the range of widths, lengths, and configurations in any given case is constrained by a number of factors. Length is constrained by the location of the crossing arteries that determine the location of interchanges and ramps. Width is constrained by the number of lanes on entry and exit legs and usually involves no more than two choices. Configuration is also the result of the number of lanes on entry and exit legs as well as the number of lanes within the segment. Changing the configuration usually involves adding a lane to one of the entry or exit legs, or both, to create different linkages.

For analysis, the geometry of the weaving segment must be fully defined. This includes the number of lanes, lane widths, shoulder clearances, the details of entry and exit gore area designs (including markings), the existence and extent of barrier lines, and the length of the segment. A sketch of the weaving segment should be drawn with all appropriate dimensions shown.

Traffic demands are usually expressed as peak hour volumes under prevailing conditions. If flow rates have been directly observed in the field, the flow rates for the worst 15-min period in the peak hour may be substituted. In this case, the peak hour factor (PHF) is implicitly 1.00.

Step 2: Adjust Volume

All equations in this chapter use flow rates under equivalent ideal conditions as input variables. Thus, demand volumes and flow rates under prevailing conditions must be converted to their ideal equivalents by using Equation 12-1:

$$v_i = \frac{V_i}{PHF \times f_{HV} \times f_p}$$

Equation 12-1

where

v_i = flow rate i under ideal conditions (pc/h);

V_i = hourly volume for flow i under prevailing conditions in vehicles per hour (veh/h);

PHF = peak hour factor;

f_{HV} = adjustment factor for heavy-vehicle presence; and

f_p = adjustment factor for driver population; the subscript for the type of flow i can take on the following values:

FF = freeway to freeway;

FR = freeway to ramp;

RF = ramp to freeway;

RR = ramp to ramp;

W = weaving; and

NW = nonweaving.

Factors f_{HV} and f_p are taken from Chapter 11, Basic Freeway Segments.

If flow rates for a 15-min period have been provided as inputs, the PHF is taken to be 1.00 in this computation. If hourly volumes are converted by using a PHF other than 1.00, there is an implicit assumption that all four component flows in the weaving segment peak during the same 15-min period of the hour. This is rarely true in the field; however, such an analysis represents a worst-case scenario.

Once demand flow rates have been established, it may be convenient to construct a weaving diagram similar to those illustrated in Exhibit 12-7 (for one-sided weaving segments) and Exhibit 12-8 (for two-sided weaving segments).

Step 3: Determine Configuration Characteristics

Several key parameters characterize the configuration of a weaving segment. These are descriptive of the segment and will be used as key variables in subsequent steps of the methodology:

LC_{MIN} = minimum rate at which weaving vehicles must change lanes to complete all weaving maneuvers successfully (lc/h); and

N_{WL} = number of lanes from which weaving maneuvers may be made with either one or no lane changes.

How these values are determined depends on whether the segment under study is a one-sided or two-sided weaving segment.

One-Sided Weaving Segments

The determination of key variables in one-sided weaving segments is illustrated in Exhibit 12-7. In one-sided segments, the two weaving movements are the ramp-to-freeway and freeway-to-ramp flows. As shown in Exhibit 12-7, the following values are established:

LC_{RF} = minimum number of lane changes that must be made by one ramp-to-freeway vehicle to execute the desired maneuver successfully, and

LC_{FR} = minimum number of lane changes that must be made by one freeway-to-ramp vehicle to execute the desired maneuver successfully.

LC_{MIN} for one-sided weaving segments is given by Equation 12-2:

Equation 12-2

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

For one-sided weaving segments, the value of N_{WL} is either 2 or 3. The determination is made by a review of the geometric design and the configuration of the segment, as illustrated in Exhibit 12-5.

Two-Sided Weaving Segments

The determination of key variables in two-sided weaving segments is illustrated in Exhibit 12-8. The unique feature of two-sided weaving segments is that only the ramp-to-ramp flow is functionally weaving. From Exhibit 12-8, the following value is established:

LC_{RR} = minimum number of lane changes that must be made by one ramp-to-ramp vehicle to execute the desired maneuver successfully.

LC_{MIN} for two-sided weaving segments is given by Equation 12-3:

Equation 12-3

$$LC_{MIN} = LC_{RR} \times v_{RR}$$

For two-sided weaving segments, the value of N_{WL} is always 0 by definition.

Step 4: Determine Maximum Weaving Length

The maximum length of a weaving segment, L_{MAX}, is based on the distance beyond which additional length does not add to capacity.

The concept of maximum length of a weaving segment is critical to the methodology. Strictly defined, *maximum length* is the length at which weaving turbulence no longer has an impact on operations within the segment, or alternatively, on the capacity of the weaving segment.

Unfortunately, depending on the selected definition, these measures can be quite different. Weaving turbulence will have an impact on operations (i.e., weaving and nonweaving vehicle speeds) for distances far in excess of those defined by when the capacity of the segment is no longer affected by weaving.

This methodology uses the second definition (based on the equivalence of capacity). If the operational definition were used, the methodology would produce capacity estimates in excess of those for a similar basic freeway segment, which is illogical. The maximum length of a weaving segment (in feet) is computed from Equation 12-4:

Equation 12-4

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566N_{WL}\right]$$

where L_{MAX} is the maximum weaving segment length (using the short length definition) and other variables are as previously defined.

As VR increases, it is expected that the influence of weaving turbulence would extend for longer distances. All values of N_{WL} are either 0 (two-sided weaving segments) or 2 or 3 (one-sided weaving segments). Having more lanes from which easy weaving lane changes can be made reduces turbulence, which in turn reduces the distance over which such turbulence affects segment capacity.

Exhibit 12-9 illustrates the sensitivity of maximum length to both VR and N_{WL}. As expected, VR has a significant impact on maximum length, as does the configuration, as indicated by N_{WL}. While the maximum lengths shown can compute to very high numbers, the highest results are well outside the calibration range of the equation (limited to about 2,800 ft), and many of the situations are improbable. Values of VR on segments with N_{WL} = 2.0 lanes rarely rise above the range of 0.40 to 0.50. While values of VR above 0.70 are technically feasible on segments with N_{WL} = 3.0 lanes, they are rare.

While the extreme values in Exhibit 12-9 are not practical, it is clear that the maximum length of weaving segments can rise to 6,000 ft or more. Furthermore, the maximum length can vary over time, as VR is not a constant throughout every demand period of the day.

Exhibit 12-9
Variation of Weaving Length Versus Volume Ratio and Number of Weaving Lanes (ft)

VR	Number of Weaving Lanes $N_{WL} = 2$	$N_{WL} = 3$
0.1	3,540	1,974
0.2	4,536	2,970
0.3	5,584	4,018
0.4	6,681	5,115
0.5	7,826	6,260
0.6	9,019	7,453
0.7	10,256	8,690
0.8	11,538	9,972

If the length of the segment is greater than L_{MAX}, it should be analyzed as separate merge and diverge ramp junctions by using the methodology in Chapter 13. Any portion falling outside the influence of the merge and diverge segments is treated as a basic freeway segment.

The value of L_{MAX} is used to determine whether continued analysis of the configuration as a weaving segment is justified:

- If $L_S < L_{MAX}$, continue to Step 5; or
- If $L_S \geq L_{MAX}$, analyze the merge and diverge junctions as separate segments by using the methodology in Chapter 13.

If the segment is too long to be considered a weaving segment, then the merge and diverge areas are treated separately. Any distance between the two falling outside the influence areas of the merge and diverge segments would be considered to be a basic freeway segment and would be analyzed accordingly.

Step 5: Determine Weaving Segment Capacity

A weaving segment's capacity is controlled by either (a) the average vehicle density reaching 43 pc/mi/ln or (b) the weaving demand flow rate exceeding a value that depends on the number of weaving lanes.

The capacity of a weaving segment is controlled by one of two conditions:

- Breakdown of a weaving segment is expected to occur when the average density of all vehicles in the segment reaches 43 pc/mi/ln; or
- Breakdown of a weaving segment is expected to occur when the total weaving demand flow rate exceeds
  - o 2,400 pc/h for cases in which N_{WL} = 2 lanes, or
  - o 3,500 pc/h for cases in which N_{WL} = 3 lanes.

The first criterion is based on the criteria listed in Chapter 11, Basic Freeway Segments, which state that freeway breakdowns occur at a density of 45 pc/mi/ln. Given the additional turbulence in a weaving segment, breakdown is expected to occur at slightly lower densities.

The second criterion recognizes that there is a practical limit to how many vehicles can actually cross each other's path without causing serious operational failures. The existence of a third lane from which weaving maneuvers can be made with two or fewer lane changes in effect spreads the impacts of turbulence across segment lanes and allows for higher weaving flows.

For two-sided weaving segments (N_{WL} = 0 lanes), no limiting value on weaving flow rate is proposed. The analysis of two-sided weaving segments is approximate with this methodology, and a density sufficient to cause a breakdown is generally reached at relatively low weaving flow rates.

Weaving Segment Capacity Determined by Density

The capacity of a weaving segment, based on reaching a density of 43 pc/mi/ln, is estimated by using Equation 12-5:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_S\right] + \left[119.8N_{WL}\right]$$

Equation 12-5

where

c_{IWL} = capacity of the weaving segment under equivalent ideal conditions, per lane (pc/h/ln), and

c_{IFL} = capacity of a basic freeway segment with the same FFS as the weaving segment under equivalent ideal conditions, per lane (pc/h/ln).

All other variables are as previously defined.

The model describes the capacity of a weaving segment in terms of the difference between the capacity of a basic freeway segment and the capacity of a weaving segment with the same FFS. Capacity decreases with *VR*, which is logical. It increases as length and number of weaving lanes N_{WL} increase. These are also logical trends, as both increasing length and a larger number of weaving lanes reduce the intensity of turbulence.

Arithmetically, it is possible to get a result in which c_{IWL} is greater than c_{IFL}. In practical terms, this will never occur. The maximum length algorithm of Step 4 was found by setting the two values equal. Thus, weaving analyses would only be undertaken in cases in which c_{IWL} is less than c_{IFL}.

The value of c_{IWL} must now be converted to a total capacity under prevailing conditions by using Equation 12-6:

$$c_W = c_{IWL} N f_{HV} f_p$$

Equation 12-6

where c_W is the capacity of the weaving segment under prevailing conditions in vehicles per hour. As with all capacities, it is stated as a flow rate for a 15-min analysis period.

Weaving Segment Capacity Determined by Weaving Demand Flows

The capacity of a weaving segment, as controlled by the maximum weaving flow rates noted previously, is found from Equation 12-7:

$$c_{IW} = \frac{2{,}400}{VR} \quad \text{for } N_{WL} = 2 \text{ lanes}$$

$$c_{IW} = \frac{3{,}500}{VR} \quad \text{for } N_{WL} = 3 \text{ lanes}$$

Equation 12-7

where c_{IW} is the capacity of all lanes in the weaving segment under ideal conditions in passenger cars per hour, and all other variables are as previously defined. This value must be converted to prevailing conditions by using Equation 12-8:

$$c_W = c_{IW} f_{HV} f_p$$

Equation 12-8

Final Determination of Capacity

The final capacity is the smaller of the two estimates of Equation 12-6 and Equation 12-8. With capacity determined, a *v/c* ratio for the weaving segment may be computed from Equation 12-9:

Equation 12-9

$$v/c = \frac{v f_{HV} f_p}{c_W}$$

Adjustment factors are used because the total demand flow rate, v, is stated for equivalent ideal conditions, while c_W is stated for prevailing conditions.

Level of Service F

LOS F occurs when demand exceeds capacity.

If v/c is greater than 1.00, demand exceeds capacity, and the segment is expected to fail, that is, have a LOS of F. If this occurs, the analysis is terminated, and LOS F is assigned. At LOS F, it is expected that queues will form within the segment, possibly extending upstream beyond the weaving segment itself. Queuing on the on-ramps that are part of the weaving segment would also be expected. Where LOS F is found to exist, the analyst is urged to use the methodology of Chapter 10, Freeway Facilities, to analyze the impacts of this on upstream and downstream segments during the analysis period and over time.

Step 6: Determine Lane-Changing Rates

The equivalent hourly rate at which weaving and nonweaving vehicles make lane changes within the weaving segment is a direct measure of turbulence. It is also a key determinant of speeds and densities within the segment, which ultimately determine the existing or anticipated LOS.

It should be noted that the lane-changing rates estimated are in terms of equivalent *passenger-car lane changes*. It is assumed that heavy-vehicle lane changes create more turbulence than passenger-car lane changes.

Three types of lane changes can be made within a weaving segment:

- *Required lane changes made by weaving vehicles*: These lane changes must be made to complete a weaving maneuver and are restricted to the physical area of the weaving segment. In Step 3, the rate at which such lane changes are made by weaving vehicles, LC_{MIN}, was determined.
- *Optional lane changes made by weaving vehicles*: These lane changes are not necessary to weave successfully. They involve weaving drivers who choose to enter the weaving segment in the outer lanes of either the freeway or ramp (assuming it has more than one lane), leave the weaving segment in an outer lane, or both. Such drivers make additional lane changes beyond those absolutely required by their weaving maneuver.
- *Optional lane changes made by nonweaving vehicles*: Nonweaving vehicles may also make lane changes within the weaving segment, but neither the configuration nor their desired origin and destination would require such lane changes. Lane changes by nonweaving vehicles are always made because the driver chooses that option.

While LC_{MIN} can be computed from the weaving configuration and the demand flow rates, additional optional lane changes made by both weaving and nonweaving vehicles add to turbulence and must be estimated by using regression-based models.

Estimating the Total Lane-Changing Rate for Weaving Vehicles

The model for predicting the total lane-changing rate for weaving vehicles is of the form LC_{MIN} plus an algorithm that predicts the additional optional lane-changing rate. These are combined so that the total lane-changing rate for weaving vehicles, including both required and optional lane changes, is as shown in Equation 12-10:

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5} N^2 (1 + ID)^{0.8}\right]$$

Equation 12-10

where

LC_W = equivalent hourly rate at which weaving vehicles make lane changes within the weaving segment (lc/h);

LC_{MIN} = minimum equivalent hourly rate at which weaving vehicles must make lane changes within the weaving segment to complete all weaving maneuvers successfully (lc/h);

L_S = length of the weaving segment, using the short length definition (ft) (300 ft is the minimum value);

N = number of lanes within the weaving segment; and

ID = interchange density (int/mi).

Equation 12-10 has several interesting characteristics. The term $L_S - 300$ implies that for weaving segments of 300 ft (or shorter), weaving vehicles only make necessary lane changes, that is, $LC_W = LC_{MIN}$. While shorter weaving segments would be an aberration, they do occasionally occur. In using Equation 12-10, however, a length of 300 ft is used for all lengths less than or equal to 300 ft.

This model is also unique in that it is the first use of interchange density in a model not involving determination of the FFS. In this edition of the HCM, however, FFS is partially based on total ramp density rather than interchange density. The two measures are, of course, related to the type of interchange involved. A full cloverleaf interchange has four ramps, while a diamond interchange has two ramps. Care must be taken when determining the value of *total ramp density* and *interchange density*, as they are different numbers.

The algorithm uses the term $1 + ID$ because the value of ID may be either more than or less than 1.00, and the power term would not act consistently on the result. In determining interchange density for a weaving segment, a distance of 3 mi upstream and 3 mi downstream of the midpoint of the weaving segment is used. The number of interchanges within the 6-mi range defined above is counted and divided by 6 to determine the interchange density. The subject weaving segment should be counted as one interchange in this computation. For additional discussion of total ramp density, consult Chapter 11.

The basic sensitivities of this model are reasonable. Weaving-vehicle lane changing increases as the length and width of the weaving segment increase. A longer, wider weaving segment simply provides more opportunities for weaving vehicles to execute lane changes. Lane changing also increases as interchange density increases. Higher interchange densities mean that there are more reasons

for drivers to make optional lane changes based upon their entry or exit at a nearby interchange.

Estimating the Lane-Changing Rate for Nonweaving Vehicles

No nonweaving driver *must* make a lane change within the confines of a weaving segment. All nonweaving vehicle lane changes are, therefore, optional. These are more difficult to predict than weaving lane changes, as the motivation for nonweaving lane changes varies widely and may not always be obvious. Such lane changes may be made to avoid turbulence, to be better positioned for a subsequent maneuver, or simply to achieve a higher average speed.

The research leading to this methodology (*10*) revealed several discontinuities in the lane-changing behavior of nonweaving vehicles within weaving segments. To identify the areas of discontinuity and to develop an estimation model for these areas, it was necessary to define a "nonweaving vehicle index," I_{NW}, as given in Equation 12-11:

Equation 12-11

$$I_{NW} = \frac{L_S \times ID \times v_{NW}}{10{,}000}$$

This index is a measure of the tendency of conditions to induce unusually large nonweaving vehicle lane-changing rates. Large nonweaving flow rates, high interchange densities, and long weaving lengths seem to produce situations in which nonweaving lane-changing rates are unusually elevated.

Two models are used to predict the rate at which nonweaving vehicles change lanes in weaving segments. The first, Equation 12-12, covers the majority of cases, that is, cases for which normal lane-changing characteristics are expected. This is the case when I_{NW} is less than or equal to 1,300:

Equation 12-12

$$LC_{NW1} = \left(0.206 v_{NW}\right) + \left(0.542 L_s\right) - \left(192.6 N\right)$$

where LC_{NW1} is the rate of lane changing per hour. The equation shows logical trends in that nonweaving lane changes increase with both nonweaving flow rate and segment length. Less expected is that nonweaving lane changing *decreases* with increasing number of lanes. This trend is statistically very strong and likely indicates more presegregation of flows in wider weaving segments. Arithmetically, Equation 12-12 can produce a negative result. Thus, the minimum value must be externally set at 0.

The second model applies to a small number of cases in which the combination of high nonweaving demand flow, high interchange density, and long segment length produce extraordinarily high nonweaving lane-changing rates. Equation 12-13 is used in cases for which I_{NW} is greater than or equal to 1,950:

Equation 12-13

$$LC_{NW2} = 2{,}135 + 0.223\left(v_{NW} - 2{,}000\right)$$

where LC_{NW2} is the lane-changing rate per hour, and all other variables are as previously defined.

Unfortunately, Equation 12-12 and Equation 12-13 are discontinuous and cover discontinuous ranges of I_{NW}. If the nonweaving index is between 1,300 and

1,950, a straight interpolation between the values of LC_{NW1} and LC_{NW2} is used as shown in Equation 12-14:

$$LC_{NW3} = LC_{NW1} + \left(LC_{NW2} - LC_{NW1}\right)\left(\frac{I_{NW} - 1{,}300}{650}\right)$$

Equation 12-14

where LC_{NW3} is the lane-changing rate per hour, and all other variables are as previously defined. Equation 12-14 only works for cases in which LC_{NW1} is less than LC_{NW2}. In the vast majority of cases, this will be true (unless the weaving length is longer than the maximum length estimated in Step 4). In the rare case when it is not true, LC_{NW2} is used.

Equation 12-15 summarizes this in a more precise way:

$$\begin{array}{ll} \text{If } I_{NW} \le 1{,}300: & LC_{NW} = LC_{NW1} \\ \text{If } I_{NW} \ge 1{,}950: & LC_{NW} = LC_{NW2} \\ \text{If } 1{,}300 < I_{NW} < 1{,}950: & LC_{NW} = LC_{NW3} \\ \text{If } LC_{NW1} \ge LC_{NW2}: & LC_{NW} = LC_{NW2} \end{array}$$

Equation 12-15

Total Lane-Changing Rate

The total lane-changing rate LC_{ALL} of all vehicles in the weaving segment, in lane changes per hour, is computed from Equation 12-16:

$$LC_{ALL} = LC_W + LC_{NW}$$

Equation 12-16

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles in Weaving Segment

The heart of this methodology is the estimation of the average speeds of weaving and nonweaving vehicles in the weaving segment. These speeds are estimated separately because they are affected by different factors, and they can be significantly different from each other.

The speeds of weaving and nonweaving vehicles will be combined to find a space mean speed of all vehicles in the segment. This will then be converted to a density, which will determine the LOS.

Average Speed of Weaving Vehicles

The algorithm for predicting the average speed of weaving vehicles in a weaving segment may be generally stated as shown in Equation 12-17:

$$S_W = S_{MIN} + \left(\frac{S_{MAX} - S_{MIN}}{1 + W}\right)$$

Equation 12-17

where

S_W = average speed of weaving vehicles within the weaving segment (mi/h),

S_{MIN} = minimum average speed of weaving vehicles expected in a weaving segment (mi/h),

S_{MAX} = maximum average speed of weaving vehicles expected in a weaving segment (mi/h), and

W = weaving intensity factor.

The form of the model is logical and constrains the results to a reasonable range defined by the minimum and maximum speed expectations. The term $1 + W$ accommodates a weaving intensity factor that can be more or less than 1.0.

For this methodology, the minimum expected speed is taken to be 15 mi/h, and the maximum expected speed is the FFS. As with all analyses, the FFS is best observed in the field, either on the subject facility or a similar facility. When measured, the FFS should be observed within the weaving segment.

In situations that require the FFS to be estimated, the model described in Chapter 11, Basic Freeway Segments, is used. The average speed of weaving vehicles within the weaving segment is estimated by using Equation 12-18 and Equation 12-19:

Equation 12-18

$$S_W = 15 + \left(\frac{FFS - 15}{1 + W} \right)$$

Equation 12-19

$$W = 0.226 \left(\frac{LC_{ALL}}{L_S} \right)^{0.789}$$

Note that weaving intensity is based on the total lane-changing rate within the weaving segment. More specifically, it is based on the hourly rate of lane changes per foot of weaving length. This might be thought of as a measure of the density of lane changes. In addition, the lane-changing rate itself depends on many demand and physical factors related to the design of the segment.

Average Speed of Nonweaving Vehicles

The average speed of nonweaving vehicles in a weaving segment is estimated by using Equation 12-20:

Equation 12-20

$$S_{NW} = FFS - (0.0072\,LC_{MIN}) - \left(0.0048 \frac{v}{N} \right)$$

Equation 12-20 treats nonweaving speed as a reduction from FFS. As would be expected, the speed is reduced as v/N increases. More interesting is the appearance of LC_{MIN} in the equation. LC_{MIN} is a measure of minimal weaving turbulence, assuming that weaving vehicles make only necessary lane changes. It depends on both the configuration of the weaving segment and weaving demand flow rates. Thus, nonweaving speeds decrease as weaving turbulence increases.

Average Speed of All Vehicles

The space mean speed of all vehicles in the weaving segment is computed by using Equation 12-21:

Equation 12-21

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W} \right) + \left(\frac{v_{NW}}{S_{NW}} \right)}$$

Step 8: Determine LOS

The LOS in a weaving segment, as in all freeway analysis, is related to the density in the segment. Exhibit 12-10 provides LOS criteria for weaving segments on freeways, collector–distributor (C-D) roadways, and multilane highways. This methodology was developed for freeway weaving segments, although an isolated C-D roadway was included in its development. The methodology may be applied to weaving segments on uninterrupted segments of multilane surface facilities, although its use in such cases is approximate.

LOS can be determined for weaving segments on freeways, multilane highways, and C-D roadways.

Exhibit 12-10
LOS for Weaving Segments

LOS	Density (pc/mi/ln)	
	Freeway Weaving Segments	Weaving Segments on Multilane Highways or C-D Roadways
A	0–10	0–12
B	>10–20	>12–24
C	>20–28	>24–32
D	>28–35	>32–36
E	>35	>36
F	Demand exceeds capacity	

The boundary between stable and unstable flow—the boundary between levels of service E and F—occurs when the demand flow rate exceeds the capacity of the segment, as described in Step 5. The threshold densities for other levels of service were set relative to the criteria for basic freeway segments (or multilane highways). In general, density thresholds in weaving segments are somewhat higher than those for similar basic freeway segments (or multilane highways). It is believed that drivers will tolerate higher densities in an area where lane-changing turbulence is expected than on basic segments.

To apply density criteria, the average speed of all vehicles, computed in Step 7, must be converted to density by using Equation 12-22.

$$D = \frac{\left(\frac{v}{N}\right)}{S}$$

Equation 12-22

where D is density in passenger cars per mile per lane and all other variables are as previously defined.

SPECIAL CASES

Multiple Weaving Segments

When a series of closely spaced merge and diverge areas creates overlapping weaving movements (between different merge–diverge pairs) that share the same segment of a roadway, a multiple weaving segment is created. In earlier editions of the HCM, a specific application of the weaving methodology for two-segment multiple weaving segments was included. While it was a logical extension of the methodology, it did not address cases in which three or more sets of weaving movements overlapped, nor was it well-supported by field data.

Multiple weaving segments should be analyzed as separate merge, diverge, and simple weaving segments, as appropriate.

Multiple weaving segments should be segregated into separate merge, diverge, and simple weaving segments, with each segment appropriately analyzed by using this chapter's methodology or that of Chapter 13, Freeway Merge and Diverge Segments. Chapter 11, Basic Freeway Segments, contains information relative to the process of identifying appropriate segments for analysis.

C-D Roadways

The methodology applies approximately to C-D roadways, but its use may produce an overly negative view of operations.

A common design practice often results in weaving movements that occur on C-D roadways that are part of a freeway interchange. The methodology of this chapter may be approximately applied to such segments. The FFS used must be appropriate to the C-D roadway. It would have to be measured on an existing or similar C-D roadway, as the predictive methodology of FFS given in Chapter 11 does not apply to such roadways. It is less clear that the LOS criteria of Exhibit 12-10 are appropriate. Many C-D roadways operate at lower speeds and higher densities than on basic segments, and the criteria of Exhibit 12-10 may produce an inappropriately negative view of operations on a C-D roadway.

If the measured FFS of a C-D roadway is high (greater than or equal to 50 mi/h), the results of analysis can be expected to be reasonably accurate. At lower FFS values, results would be more approximate.

Multilane Highways

Multilane highway weaving segments may be analyzed with this methodology, except in the vicinity of signalized intersections.

Weaving segments may occur on surface multilane highways. As long as such segments are a sufficient distance away from signalized intersections—so that platoon movements are not an issue—the methodology of this chapter may be approximately applied.

Arterial Weaving

No generally accepted analysis methodologies currently exist for arterial weaving movements.

The methodology of this chapter does not apply to weaving segments on arterials. Arterial weaving is strongly affected by the proximity and timing of signals along the arterial. At the present time, there are no generally accepted analytic methodologies for analyzing weaving movements on arterials.

4. APPLICATIONS

The methodology of this chapter is most often used to estimate the capacity and LOS of freeway weaving segments. The steps are most easily applied in the operational analysis mode, that is, all traffic and roadway conditions are specified, and a solution for the capacity (and v/c ratio) is found along with an expected LOS. Other types of analysis, however, are possible.

DEFAULT VALUES

An NCHRP report (*10*) provides a comprehensive presentation of potential default values for uninterrupted-flow facilities. Default values for freeways are summarized in Chapter 10, Freeway Facilities. These defaults cover the key characteristics of PHF and percentage of heavy vehicles. Recommendations are based on geographical region, population, and time of day. All general freeway default values may be applied to the analysis of weaving segments in the absence of field data or projected conditions.

There are many specific variables related to weaving segments. It is, therefore, virtually impossible to specify default values of such characteristics as length, width, configuration, and balance of weaving and nonweaving flows. Weaving segments are a detail of the freeway design and should therefore be treated only with the specific characteristics of the segment known or projected. Small changes in some of these variables can and do yield significant changes in the analysis results.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational, design, and planning and preliminary engineering.

Operational Analysis

The methodology of this chapter is most easily applied in the operational analysis mode. In this application, all weaving demands and geometric characteristics are known, and the output of the analysis is the expected LOS and the capacity of the segment. Secondary outputs include the average speed of component flows, the overall density in the segment, and measures of lane-changing activity.

Design Analysis

Design analysis is best accomplished by iterative operational analyses on a small number of candidate designs.

In design applications, the desired output is the length, width, and configuration of a weaving segment that will sustain a target LOS for given demand flows. This application is best accomplished by iterative operational analyses on a small number of candidate designs.

Generally, there is not a great deal of flexibility in establishing the length and width of a segment, and only limited flexibility in potential configurations. The location of intersecting facilities places logical limitations on the length of the weaving segment. The number of entry and exit lanes on ramps and the freeway itself limits the number of lanes to, at most, two choices. The entry and exit

design of ramps and the freeway facility also produces a configuration that can generally only be altered by adding or subtracting a lane from an entry or exit roadway. Thus, iterative analyses of candidate designs are relatively easy to pursue, particularly with the use of HCM-replicating software.

Planning and Preliminary Engineering

Planning and preliminary engineering applications generally have the same desired outputs as design applications: the geometric design of a weaving segment that can sustain a target LOS for specified demand flows.

In the planning and preliminary design phase, however, demand flows are generally stated as average annual daily traffic (AADT) statistics that must be converted to directional design hour volumes. A number of variables may be unknown (e.g., PHF and percentage of heavy vehicles); these may be replaced by default values.

Service Flow Rates, Service Volumes, and Daily Service Volumes

This manual defines three sets of values that are related to LOS boundary conditions:

SF_i = service flow rate for LOS *i* (veh/h),

SV_i = service volume for LOS *i* (veh/h), and

DSV_i = daily service volume for LOS *i* (veh/day).

The *service flow rate* is the maximum rate of flow (for a 15-min interval) that can be accommodated on a segment while maintaining all operational criteria for LOS *i* under prevailing roadway and traffic conditions. The *service volume* is the maximum hourly volume that can be accommodated on a segment while maintaining all operational criteria for LOS *i* during the worst 15 min of the hour under prevailing roadway and traffic conditions. The *daily service volume* is the maximum AADT that can be accommodated on a segment while maintaining all operational criteria for LOS *i* during the worst 15 min of the peak hour under prevailing roadway and traffic conditions. The service flow rate and service volume are unidirectional values, while the daily service volume is a total two-way volume. In the context of a weaving section, the daily service volume is highly approximate, as it is rare that both directions of a freeway have a weaving segment with similar geometry.

In general, service flow rates are initially computed for ideal conditions and are then converted to prevailing conditions by using Equation 12-23 and the appropriate adjustment factors from Chapter 11, Basic Freeway Segments:

Equation 12-23

$$SF_i = SFI_i \times f_{HV} \times f_p$$

where

SFI_i = service flow rate under ideal conditions (pc/h),

f_{HV} = adjustment factor for heavy-vehicle presence (Chapter 11), and

f_p = adjustment factor for driver population (Chapter 11).

The methodology of this chapter is used to determine the values of ideal service flow rate (*SFI*) for the specific weaving segment under study. The capacity of the segment is equivalent to the ideal service flow rate for LOS E. For other levels of service, the total flow rates required to produce threshold densities (Exhibit 12-10) are found. This is an iterative procedure in which all other characteristics are held constant. Iterative analyses are conducted until the defining densities are produced.

Once the ideal service flow rates are determined, service flow rates under prevailing conditions are computed by using Equation 12-23. These can be converted to hourly service volumes *SV* by using Equation 12-24. Service volumes can then be converted to daily service volumes *DSV* by using Equation 12-25.

Equation 12-24

$$SV_i = SF_i \times PHF$$

Equation 12-25

$$DSV_i = \frac{SV_i}{K \times D}$$

where

K = proportion of AADT occurring during the peak hour, and

D = proportion of traffic in the peak direction.

All other variables are as previously defined.

Example Problem 5 illustrates the computation of service flow rates, service volumes, and daily service volumes for a specific weaving segment.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for the application of alternative tools to the analysis of freeway weaving segments. Additional information on this topic, including supplemental example problems, may be found in Chapter 27, Freeway Weaving: Supplemental, located in Volume 4.

Strengths of the HCM Procedure

The procedures in this chapter were developed from extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent a body of expert consensus. Most alternative tools will not include the level of detail present in this methodology concerning the weaving configuration and balance of weaving demand flows.

Specific strengths of the HCM procedure include

- Providing capacity estimates for specific weaving configurations as a function of various input parameters, which current simulators do not provide directly (and in some cases may require as an input);
- Considering geometric characteristics (such as lane widths) in more detail than most simulation algorithms;

- Producing a single deterministic estimate of LOS, which is important for some purposes, such as development impact reviews; and
- Generating reproducible results with a small commitment of resources (including calibration) from a precisely documented methodology.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

Weaving segments can be analyzed by using a variety of stochastic and deterministic simulation tools that address freeways. These tools can be very useful in analyzing the extent of congestion when there are failures within the simulated facility range and when interaction with other freeway segments and other facilities is present.

The limitations stated earlier in this chapter may be addressed by using available simulation tools. The following conditions, which are beyond the scope of this chapter, are treated explicitly by simulation tools:

- *Managed lanes within the weaving segment.* These lanes are typically modeled explicitly by simulation; for example, when one or more weaving movements are regulated by using pavement markings, signage, physical longitudinal barriers, or some combination of these.
- *Ramp metering on entrance ramps forming part of the weaving segment.* These features are also modeled explicitly by many tools.
- *Specific operating conditions when oversaturated conditions exist.* In this case, it is necessary to ensure that both the spatial and the temporal boundaries of the analysis extend beyond the congested operation.
- *Effects of intelligent transportation system technologies on weaving segment operations.* Some intelligent transportation system features such as dynamic message signs are offered by a few simulation tools. Some features are modeled explicitly by simulation; others may be approximated by using assumptions (e.g., by modifying origin–destination demands by time interval).
- *Multiple weaving segments.* Multiple weaving segments were removed from this edition of the manual. They may be addressed to some extent by the procedures given in Chapter 10 for freeway facilities. Complex combinations of weaving segments may be analyzed more effectively by simulation tools, although such analyses might require extensive calibration of origin–destination characteristics.

Because of the interactions between adjacent freeway segments, alternative tools will find their principal application to freeways containing weaving segments at the facility level and not to isolated freeway weaving segments.

Additional Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the speed and density in a weaving segment given traffic demands from both the weaving and the nonweaving movements. Capacity estimates and maximum weaving lengths are

also produced. Alternative tools offer additional performance measures including delay, stops, queue lengths, fuel consumption, pollution, and operating costs.

In addition to offering more performance measures, alternative tools can identify specific point problems that could be overlooked in a segment-level analysis.

As with most other procedural chapters in this manual, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of queuing caused by capacity constraints on the exit ramp of a weaving segment, including difficulty in making the required lane changes, is a good example of a situation that can benefit from the increased insight offered by a microscopic model. An example of the effect of exit ramp queue backup is presented in Chapter 27, Freeway Weaving: Supplemental.

Development of HCM-Compatible Performance Measures Using Alternative Tools

When using alternative tools, the analyst must be careful to note the definitions of simulation outputs. The principal measures involved in the performance analysis of weaving segments are speed and delay. These terms are generally defined in the same manner by alternative tools; however, there are subtle differences among tools that often make it difficult to apply HCM criteria directly to the outputs of other tools. Performance measure comparisons are discussed in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Conceptual differences between the HCM and stochastic simulation models make direct comparison difficult for weaving segments. The HCM uses a set of deterministic equations developed and calibrated with field data. Simulation models treat each vehicle as a separate object to be propagated through the system. The physical and behavioral characteristics of drivers and vehicles in the HCM are represented in deterministic equations that compute passenger car equivalences, lane-changing rates, maximum weaving lengths, capacity, speed, and density. Simulation models apply the characteristics to each driver and vehicle, and these characteristics produce interactions between vehicles, the sum total of which determines the performance measures for a weaving segment.

Direct comparison of the numerical outputs from the HCM and alternative tools can be misleading.

One good example of the difference between microscopic and macroscopic modeling is how trucks are entered into the models. The HCM uses a conversion factor that increases the demand volumes to reflect the proportion of trucks. Simulation models deal with trucks explicitly by assigning more sluggish characteristics to each of them. The result is that HCM capacities, densities, and so forth are expressed in equivalent passenger car units, whereas the corresponding simulation values are represented by actual vehicles.

The HCM methodology estimates the speeds of weaving and nonweaving traffic streams, and on the basis of these estimates it determines the density within the weaving segment. Simulators that provide outputs on a link-by-link basis do not differentiate between weaving and nonweaving movements within

a given link; thus, comparing these (intermediate) results to other tools would be somewhat difficult.

For a given set of inputs, simulation tools should produce answers that are similar to each other and to the HCM. Although most differences should be reconcilable through calibration and identification of point problems within a segment, precise numerical agreement is not generally a reasonable expectation.

Sample Calculations Illustrating Alternative Tool Applications

Supplemental computational examples illustrating the use of alternative tools are included in Chapter 27 of Volume 4.

Chapter 27, Freeway Weaving: Supplemental, contains three examples that illustrate the application of alternative tools to freeway weaving segments. All of the problems are based on Example Problem 1 presented later in this chapter. Three questions are addressed by using a typical simulation tool:

1. Can the weaving segment capacity be estimated realistically by simulation by varying the demand volumes up to and beyond capacity?
2. How does the demand affect the performance in terms of speed and density in the weaving segment when the default model parameters are used for vehicle and behavioral characteristics?
3. How would the queue backup from a signal at the end of the off-ramp affect the weaving operation?

5. EXAMPLE PROBLEMS

Exhibit 12-11
List of Example Problems

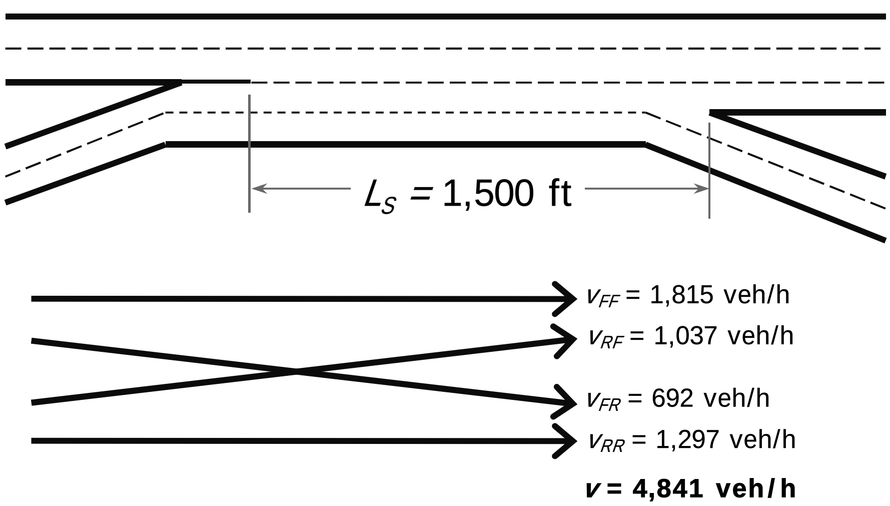
Example Problem	Description	Application
1	LOS of a major weaving segment	Operational Analysis
2	LOS of a ramp-weaving segment	Operational Analysis
3	LOS of a two-sided weaving segment	Operational Analysis
4	Design of a major weaving segment for a desired LOS	Design
5	Service volume table construction	Service Volumes

EXAMPLE PROBLEM 1: LOS OF A MAJOR WEAVING SEGMENT

The Weaving Segment

The subject of this operational analysis is a major weaving segment on an urban freeway, as shown in Exhibit 12-12.

Exhibit 12-12
Major Weaving Segment for Example Problem 1



What is the LOS and capacity of the weaving segment shown in Exhibit 12-12?

The Facts

In addition to the information contained in Exhibit 12-12, the following characteristics of the weaving segment are known:

PHF = 0.91 (for all movements);

Heavy vehicles = 10% trucks, 0% recreational vehicles (RVs) (all movements);

Driver population = regular commuters;

FFS = 65 mi/h;

c_{IFL} = 2,350 pc/h/ln (for FFS = 65 mi/h);

ID = 0.8 int/mi; and

Terrain = level.

Comments

Chapter 11, Basic Freeway Segments, must be consulted to find appropriate values for the heavy-vehicle adjustment factor f_{HV} and the driver population adjustment factor f_p.

All input parameters have been specified, so default values are not needed. Demand volumes are given in vehicles per hour under prevailing conditions. These must be converted to passenger cars per hour under equivalent ideal conditions for use in equations of the methodology. The length of the segment must be compared with the maximum length for weaving analysis to determine whether the methodology of this chapter is applicable. The capacity of the weaving segment is estimated and compared with the total demand flow to determine whether LOS F exists. Lane-changing rates are estimated to allow speed estimates to be made for weaving and nonweaving flows. An average overall speed and density are computed and compared with the criteria of Exhibit 12-10 to determine LOS.

Step 1: Input Data

All inputs have been specified in Exhibit 12-12 and the Facts section of the problem statement.

Step 2: Adjust Volume

Equation 12-1 is used to convert the four component demand volumes to flow rates under equivalent ideal conditions. Chapter 11 is consulted to obtain a value of E_T (1.5 for level terrain) and f_p (1.00 for regular commuters). The heavy-vehicle adjustment factor is computed as

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_{RV}(E_{RV} - 1)} = \frac{1}{1 + 0.10(1.5 - 1)} = 0.952$$

Equation 12-1 is now used to convert all demand volumes:

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

$$v_{FF} = \frac{1{,}815}{0.91 \times 0.952 \times 1} = 2{,}094 \text{ pc/h}$$

$$v_{FR} = \frac{692}{0.91 \times 0.952 \times 1} = 798 \text{ pc/h}$$

$$v_{RF} = \frac{1{,}037}{0.91 \times 0.952 \times 1} = 1{,}197 \text{ pc/h}$$

$$v_{RR} = \frac{1{,}297}{0.91 \times 0.952 \times 1} = 1{,}497 \text{ pc/h}$$

Then

$$v_W = 798 + 1{,}197 = 1{,}995 \text{ pc/h}$$

$$v_{NW} = 2{,}094 + 1{,}497 = 3{,}591 \text{ pc/h}$$

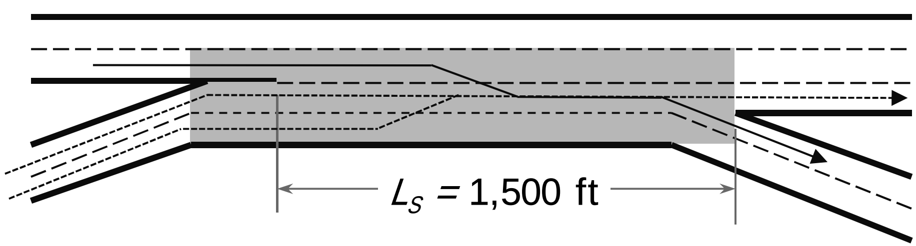
$$v = 1{,}995 + 3{,}591 = 5{,}586 \text{ pc/h}$$

$$VR = \frac{1{,}995}{5{,}586} = 0.357$$

Step 3: Determine Configuration Characteristics

The configuration is examined to determine the values of LC_{RF}, LC_{FR}, and N_{WL}. These determinations are illustrated in Exhibit 12-13. From these values, the minimum number of lane changes by weaving vehicles, LC_{MIN}, is then computed by using Equation 12-2.

Exhibit 12-13
Determination of Configuration Variables for Example Problem 1



From Exhibit 12-13, it can be seen that ramp-to-freeway vehicles can execute their weaving maneuver without making a lane change (if they so desire). Thus, $LC_{RF} = 0$. Freeway-to-ramp vehicles must make at least one lane change to complete their desired maneuver. Thus, $LC_{FR} = 1$. If optional lane changes are considered, weaving movements can be accomplished with one or no lane changes from both entering ramp lanes and from the rightmost freeway lane. Thus, $N_{WL} = 3$. Equation 12-2 can now be employed:

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR}) = (0 \times 1197) + (1 \times 798) = 798 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length

The maximum length over which weaving movements may exist is determined by Equation 12-4. The determination is case specific, and the result is valid only for the case under consideration:

$$L_{MAX} = \left[5728(1 + VR)^{1.6}\right] - \left[1566 N_{WL}\right]$$

$$L_{MAX} = \left[5728(1 + 0.357)^{1.6}\right] - \left[1566 \times 3\right] = 4{,}639 \text{ ft}$$

As the maximum length is significantly greater than the actual segment length of 1,500 ft, weaving operations do exist, and the analysis may continue with the weaving analysis methodology.

Step 5: Determine Weaving Segment Capacity

Capacity may be controlled by one of two factors: operations reaching a maximum density of 43 pc/mi/ln or by the weaving demand flow rate reaching

3,500 pc/h (for a weaving segment with N_{WL} = 3). Equation 12-5 through Equation 12-8 are used to make these determinations.

Capacity Controlled by Density

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_s\right] + \left[119.8N_{WL}\right]$$

$$c_{IWL} = 2{,}350 - \left[438.2(1+0.357)^{1.6}\right] + \left[0.0765 \times 1{,}500\right] + \left[119.8 \times 3\right]$$

$$c_{IWL} = 2{,}110 \text{ pc/h/ln}$$

$$c_W = c_{IWL} N f_{HV} f_p = 2{,}110 \times 4 \times 0.952 \times 1 = 8{,}038 \text{ veh/h}$$

Capacity Controlled by Maximum Weaving Flow Rate

$$c_{IW} = \frac{3{,}500}{VR} = \frac{3{,}500}{0.357} = 9{,}800 \text{ pc/h}$$

$$c_W = c_{IW} f_{HV} f_p = 9{,}800 \times 0.952 \times 1 = 9{,}333 \text{ veh/h}$$

Note that the methodology computes the capacity controlled by density in passenger cars per hour per lane, while the capacity controlled by maximum weaving flow rate is computed in passenger cars per hour. After conversion, however, both are in units of vehicles per hour.

The controlling value is the smaller of these, or 8,038 veh/h. As the total demand flow rate is only 5,320 veh/h, the capacity is clearly sufficient, and this situation will not result in LOS F.

Step 6: Determine Lane-Changing Rates

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}(N^2)(1+ID)^{0.8}\right]$$

$$LC_W = 798 + 0.39\left[(1{,}500 - 300)^{0.5} 4^2 (1+0.8)^{0.8}\right] = 1{,}144 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{1{,}500 \times 0.8 \times 3{,}591}{10{,}000} = 431 < 1{,}300$$

$$LC_{NW} = (0.206 v_{NW}) + (0.542 L_S) - (192.6N)$$

$$LC_{NW} = (0.206 \times 3{,}591) + (0.542 \times 1{,}500) - (192.6 \times 4) = 782 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}144 + 782 = 1{,}926 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{1{,}926}{1{,}500}\right)^{0.789} = 0.275$$

Then

$$S_W = 15 + \left(\frac{FFS - 15}{1 + W}\right) = 15 + \left(\frac{65 - 15}{1 + 0.275}\right) = 54.2 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048\frac{v}{N}\right)$$

$$S_{NW} = 65 - (0.0072 \times 798) - \left(0.0048 \times \frac{5{,}586}{4}\right) = 52.5 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{3{,}591 + 1{,}995}{\left(\frac{3{,}591}{52.5}\right) + \left(\frac{1{,}995}{54.2}\right)} = 53.1 \text{ mi/h}$$

Step 8: Determine LOS

Equation 12-22 is used to convert the average speed of all vehicles in the segment to an average density:

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{5{,}586}{4}\right)}{53.1} = 26.3 \text{ pc/mi/ln}$$

The resulting density of 26.3 pc/mi/ln is compared with the LOS criteria of Exhibit 12-10. The LOS is C, as the density is within the specified range of 20 to 28 pc/h/ln for that level.

Discussion

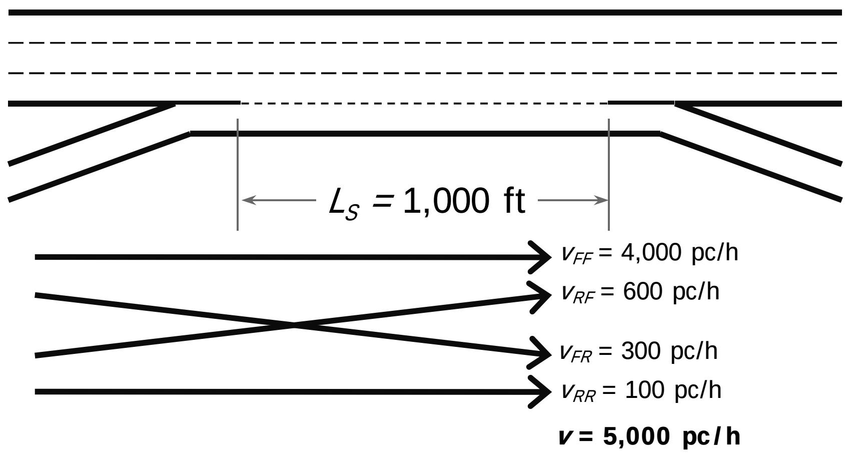
As indicated by the results, this weaving segment operates at LOS C, with an average speed of 53.1 mi/h for all vehicles. Weaving vehicles travel a bit faster than nonweaving vehicles, primarily because the configuration favors weaving vehicles, allowing many weaving maneuvers to be made without making a lane change. The demand flow rate of 4,841 veh/h is considerably less than the capacity of the segment, 8,038 veh/h. In other words, demand can grow significantly before reaching the capacity of the segment.

EXAMPLE PROBLEM 2: LOS OF A RAMP-WEAVING SEGMENT

The Weaving Segment

The weaving segment that is the subject of this operational analysis is shown in Exhibit 12-14. It is a typical ramp-weave segment.

Exhibit 12-14
Ramp-Weave Segment for Example Problem 2



What is the capacity of the weaving segment of Exhibit 12-14, and at what LOS is it expected to operate with the demand flow rates as shown?

The Facts

In addition to the information given in Exhibit 12-14, the following facts are known about the subject weaving segment:

PHF	=	1.00 (demands stated as flow rates);
Heavy vehicles	=	0% trucks, 0% RVs (demands given as passenger car equivalents);
Driver population	=	regular commuters;
FFS	=	75 mi/h;
c_{IFL}	=	2,400 pc/h/ln (for FFS = 75 mi/h);
ID	=	1.0 int/mi; and
Terrain	=	level.

Comments

Because the demands have been specified as flow rates in passenger cars per hour under equivalent ideal conditions, Chapter 11 does not have to be consulted to obtain appropriate adjustment factors.

Several of the computational steps related to converting demand volumes to flow rates under equivalent ideal conditions are trivial, as demands are already specified in that form. Lane-changing characteristics will be estimated. The maximum length for weaving operations in this case will be estimated and compared with the actual length of the segment. The capacity of the segment will be estimated and compared with the demand to determine whether LOS F exists. If it does not, component flow speeds will be estimated and averaged. A density

will be estimated and compared with the criteria of Exhibit 12-10 to determine the expected LOS.

Step 1: Input Data

All input data are stated in Exhibit 12-14 and the Facts section.

Step 2: Adjust Volume

Because all demands are stated as flow rates in passenger cars per hour under equivalent ideal conditions, no further conversions are necessary. Key volume parameters are as follows:

$$v_{FF} = 4{,}000 \text{ pc/h}$$

$$v_{FR} = 600 \text{ pc/h}$$

$$v_{RF} = 300 \text{ pc/h}$$

$$v_{RR} = 100 \text{ pc/h}$$

$$v_W = 600 + 300 = 900 \text{ pc/h}$$

$$v_{NW} = 4{,}000 + 100 = 4{,}100 \text{ pc/h}$$

$$v = 4{,}100 + 900 = 5{,}000 \text{ pc/h}$$

$$VR = \frac{900}{5{,}000} = 0.180$$

Step 3: Determine Configuration Characteristics

The configuration is examined to determine the values of LC_{RF}, LC_{FR}, and N_{WL}. These determinations are illustrated in Exhibit 12-15. From these values, the minimum number of lane changes by weaving vehicles LC_{MIN} is then computed by using Equation 12-2.

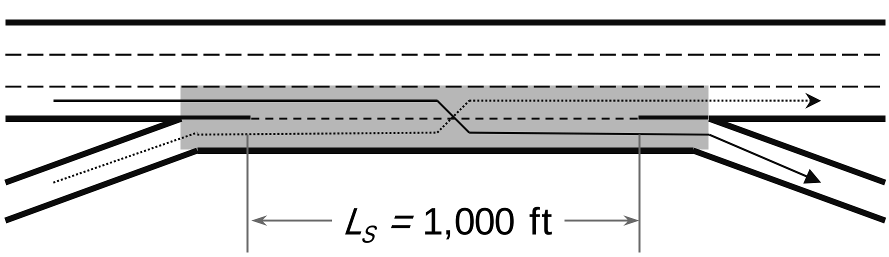


Exhibit 12-15
Configuration Characteristics for Example Problem 2

From Exhibit 12-15, it is clear that all ramp-to-freeway vehicles must make at least one lane change ($LC_{RF} = 1$), and all freeway-to-ramp vehicles must make at least one lane change ($LC_{FR} = 1$). It is also clear that a weaving maneuver can only be completed with a single lane change from the right lane of the freeway or the auxiliary lane ($N_{WL} = 2$). Then, by using Equation 12-2, LC_{MIN} is computed as

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

$$LC_{MIN} = (1 \times 600) + (1 \times 300) = 900 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length

The maximum length over which weaving operations may exist for the segment described is found by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.180)^{1.6}\right] - \left[1{,}566 \times 2\right] = 4{,}333 \text{ ft} > 1{,}000 \text{ ft}$$

As the maximum length for weaving operations significantly exceeds the actual length, this is a weaving segment, and the analysis continues.

Step 5: Determine Weaving Segment Capacity

The capacity of the weaving segment is controlled by one of two limiting factors: density reaches 43 pc/mi/ln or weaving demand reaches 2,400 pc/h for the configuration of Exhibit 12-15.

Capacity Limited by Density

The capacity limited by reaching a density of 43 pc/mi/ln is estimated by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765 L_s\right] + \left[119.8 N_{WL}\right]$$

$$c_{IWL} = 2{,}400 - \left[438.2(1+0.180)^{1.6}\right] + \left[0.0765 \times 1{,}000\right] + \left[119.8 \times 2\right]$$

$$c_{IWL} = 2{,}145 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 2{,}145 \times 4 \times 1 \times 1 = 8{,}580 \text{ pc/h}$$

Capacity Limited by Weaving Demand Flow

The capacity limited by the weaving demand flow is estimated by using Equation 12-7 and Equation 12-8:

$$c_{IW} = \frac{2{,}400}{VR} = \frac{2{,}400}{0.180} = 13{,}333 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} \times f_p = 13{,}333 \times 1 \times 1 = 13{,}333 \text{ pc/h}$$

The controlling capacity is the smaller value, or 8,580 pc/h. At this point, the value is usually stated as vehicles per hour. In this case, because inputs were already adjusted and were stated in passenger cars per hour, conversions back to vehicles per hour are not possible.

As the capacity is larger than the demand flow rate of 5,000 pc/h, LOS F does not exist, and the analysis continues.

Step 6: Determine Lane-Changing Rates

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}\left(N^2\right)(1 + ID)^{0.8}\right]$$

$$LC_W = 900 + 0.39\left[(1{,}000 - 300)^{0.5} 4^2 (1+1)^{0.8}\right] = 1{,}187 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{1{,}000 \times 1 \times 4{,}100}{10{,}000} = 410 < 1{,}300$$

$$LC_{NW} = (0.206 v_{NW}) + (0.542 L_S) - (192.6N)$$

$$LC_{NW} = (0.206 \times 4{,}100) + (0.542 \times 1{,}000) - (192.6 \times 4) = 616 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}187 + 616 = 1{,}803 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20:

$$W = 0.226\left(\frac{LC_{ALL}}{L_s}\right)^{0.789} = 0.226\left(\frac{1{,}803}{1{,}000}\right)^{0.789} = 0.360$$

Then

$$S_w = 15 + \left(\frac{FFS - 15}{1 + W}\right) = 15 + \left(\frac{75 - 15}{1 + 0.360}\right) = 59.1 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048\frac{v}{N}\right)$$

$$S_{NW} = 75 - (0.0072 \times 900) - \left(0.0048 \times \frac{5{,}000}{4}\right) = 62.5 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{4{,}100 + 900}{\left(\frac{4{,}100}{62.5}\right) + \left(\frac{900}{59.1}\right)} = 61.9 \text{ mi/h}$$

Step 8: Determine LOS

The average density in the weaving segment is estimated by using Equation 12-22.

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{5{,}000}{4}\right)}{61.9} = 20.2 \text{ pc/mi/ln}$$

From Exhibit 12-10, this density is within the stated boundaries of LOS C (20 to 28 pc/mi/ln). It is, however, very close to the LOS B boundary condition.

Discussion

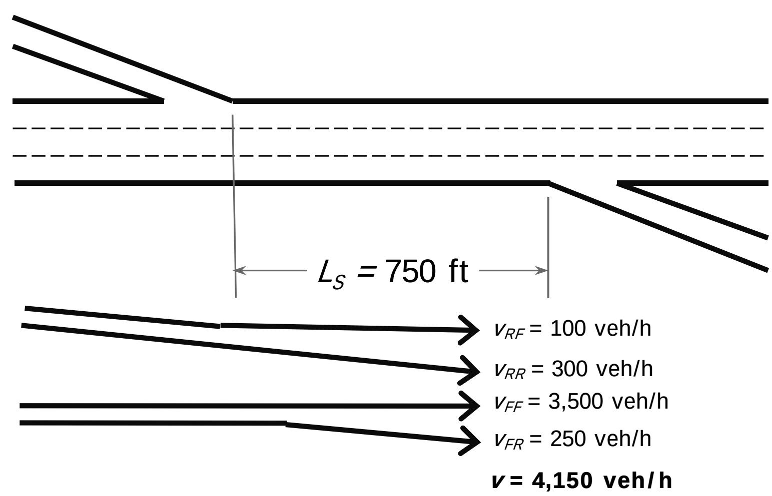
As noted, the segment is operating quite well (LOS C) and is very close to the LOS B boundary. Weaving and nonweaving speeds are relatively high, suggesting a stable flow. The demand flow rate of 5,000 pc/h is well below the capacity of the segment (8,580 pc/h). Weaving vehicles travel somewhat more slowly than nonweaving vehicles, which is typical of ramp-weave segments, where the vast majority of nonweaving vehicles are running from freeway to freeway.

EXAMPLE PROBLEM 3: LOS OF A TWO-SIDED WEAVING SEGMENT

The Weaving Segment

The weaving segment that is the subject of this example problem is shown in Exhibit 12-16.

Exhibit 12-16
Weaving Segment for Example Problem 3



What is the expected LOS and capacity for the weaving segment of Exhibit 12-16?

The Facts

In addition to the information contained in Exhibit 12-16, the following facts concerning the weaving segment are known:

PHF = 0.94 (all movements);

Heavy vehicles = 15% trucks, 0% RVs (all movements);

Driver population = regular commuters;

FFS = 60 mi/h;

c_{IFL} = 2,300 pc/h/ln (for FFS = 60 mi/h);
ID = 2 int/mi; and
Terrain = rolling.

Comments

Because this example illustrates the analysis of a two-sided weaving segment, several key parameters are redefined.

In a two-sided weaving segment, only the ramp-to-ramp flow is considered to be a weaving flow. While the freeway-to-freeway flow technically weaves with the ramp-to-ramp flow, the operation of freeway-to-freeway vehicles more closely resembles that of nonweaving vehicles. These vehicles generally make very few lane changes as they move through the segment in a freeway lane. This segment is in a busy urban corridor with a high interchange density and a relatively low FFS for the freeway.

Solution steps are the same as in the first two example problems. However, since the segment is a two-sided weaving segment, some of the key values will be computed differently as described in the methodology.

Component demand volumes will be converted to equivalent flow rates in passenger cars per hour under ideal conditions, and key demand parameters will be calculated. A maximum weaving length will be estimated to determine whether a weaving analysis is appropriate. The capacity of the weaving segment will be estimated to determine whether LOS F exists. If not, lane-changing parameters, speeds, density, and LOS will be estimated.

Step 1: Input Data

All information concerning this example problem is given in Exhibit 12-16 and the Facts section.

Step 2: Adjust Volume

To convert demand volumes to flow rates under equivalent ideal conditions, Chapter 11 must be consulted to obtain the following values:

E_T = 2.5 (for rolling terrain)
f_p = 1.0 (for regular commuters)

Then

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.15(2.5 - 1)} = 0.816$$

Component demand volumes may now be converted to flow rates under equivalent ideal conditions:

$$v_{FF} = \frac{3{,}500}{0.94 \times 0.816 \times 1} = 4{,}561 \text{ pc/h}$$

$$v_{FR} = \frac{250}{0.94 \times 0.816 \times 1} = 326 \text{ pc/h}$$

$$v_{RF} = \frac{100}{0.94 \times 0.816 \times 1} = 130 \text{ pc/h}$$

$$v_{RR} = \frac{300}{0.94 \times 0.816 \times 1} = 391 \text{ pc/h}$$

Because this is a two-sided weaving segment, the only weaving flow is the ramp-to-ramp flow. All other flows are treated as nonweaving. Then

v_W = 391 pc/h

v_{NW} = 4,561 + 326 + 130 = 5,017 pc/h

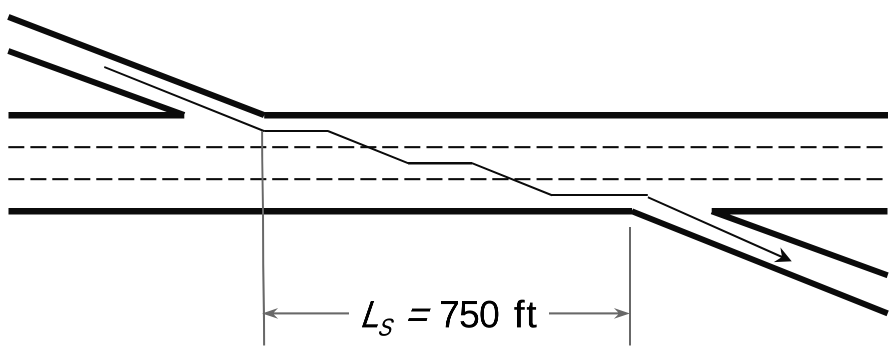
v = 5,017 + 391 = 5,408 pc/h

VR = 391/5,408 = 0.072

Step 3: Determine Configuration Characteristics

The determination of configuration characteristics is also affected by the existence of a two-sided weaving segment. Exhibit 12-17 illustrates the determination of LC_{RR}, the key variable for two-sided weaving segments. For such segments, N_{WL} = 0 by definition.

Exhibit 12-17
Configuration Characteristics for Example Problem 3



From Exhibit 12-17, ramp-to-ramp vehicles must make two lane changes to complete their desired weaving maneuver. Then

$$LC_{MIN} = LC_{RR} \times v_{RR} = 2 \times 391 = 782 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length

The maximum length of a weaving segment for this configuration and demand scenario is estimated by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.072)^{1.6}\right] - \left[1{,}566 * 0\right] = 6{,}405 \text{ ft} > 750 \text{ ft}$$

In this two-sided configuration, the impacts of weaving on operations could be felt at lengths as long as 6,405 ft. As this is significantly greater than the actual length of 750 ft, this segment clearly operates as a weaving segment and, therefore, the methodology of this chapter should be applied.

Step 5: Determine Weaving Segment Capacity

The capacity of a two-sided weaving segment can only be estimated when a density of 43 pc/h/ln is reached. This estimation is made by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_s\right] + \left[119.8N_{WL}\right]$$

$$c_{IWL} = 2{,}300 - \left[438.2(1+0.072)^{1.6}\right] + \left[0.0765\times 750\right] + \left[119.8\times 0\right]$$

$$c_{IWL} = 1{,}867 \text{ pc/h/ln}$$

$$c_W = c_{IWL}\times N\times f_{HV}\times f_p = 1{,}867\times 3\times 0.816\times 1 = 4{,}573 \text{ veh/h} > 4{,}150 \text{ veh/h}$$

Because the capacity of the segment exceeds the demand volume (in vehicles per hour), LOS F is not expected, and the analysis may be continued.

Step 6: Determine Lane-Changing Rates

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}\left(N^2\right)(1+ID)^{0.8}\right]$$

$$LC_W = 782 + 0.39\left[(750-300)^{0.5}3^2(1+2)^{0.8}\right] = 961 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{750\times 2\times 5{,}017}{10{,}000} = 753 < 1{,}300$$

$$LC_{NW} = \left(0.206v_{NW}\right) + \left(0.542L_s\right) - \left(192.6N\right)$$

$$LC_{NW} = \left(0.206\times 5{,}017\right) + \left(0.542\times 750\right) - \left(192.6\times 3\right) = 862 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 961 + 862 = 1{,}823 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{1{,}823}{750}\right)^{0.789} = 0.456$$

Then

$$S_W = 15 + \left(\frac{FFS-15}{1+W}\right) = 15 + \left(\frac{60-15}{1+0.456}\right) = 45.9 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048 \frac{v}{N}\right)$$

$$S_{NW} = 75 - (0.0072 \times 782) - \left(0.0048 \times \frac{5{,}408}{3}\right) = 45.7 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{5{,}017 + 391}{\left(\frac{5{,}017}{45.7}\right) + \left(\frac{391}{45.9}\right)} = 45.7 \text{ mi/h}$$

Step 8: Determine LOS

The average density in this two-sided weaving segment is estimated by using Equation 12-22:

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{5{,}408}{3}\right)}{45.7} = 39.4 \text{ pc/mi/ln}$$

From Exhibit 12-10, this density is clearly in LOS E. It is not far from the 43 pc/h/ln that would likely cause a breakdown.

Discussion

This two-sided weaving segment operates at LOS E, not far from the LOS E/F boundary. The v/c ratio is 4,150/4,573 = 0.91. The major problem is that 300 veh/h crossing the freeway from ramp to ramp creates a great deal of turbulence in the traffic stream and limits capacity. Two-sided weaving segments do not operate well with such large numbers of ramp-to-ramp vehicles. If this were a basic freeway segment, the per lane flow rate of 5,408/3 = 1,803 pc/h/ln would not be considered excessive and would be well within a basic freeway segment's capacity of 2,300 pc/h/ln.

EXAMPLE PROBLEM 4: DESIGN OF A MAJOR WEAVING SEGMENT FOR A DESIRED LOS

The Weaving Segment

A weaving segment is to be designed between two major junctions in which two urban freeways join and then separate as shown in Exhibit 12-18. Entry and exit legs have the numbers of lanes shown. The maximum length of the weaving segment is 1,000 ft, based on the location of the junctions. The FFS of all entry and exit legs is 75 mi/h. All demands are shown as flow rates under equivalent ideal conditions.

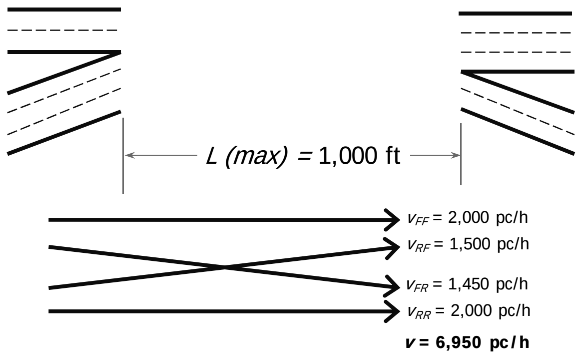


Exhibit 12-18
Weaving Segment for Example Problem 4

What design would be appropriate to deliver LOS C for the demand flow rates shown?

The Facts

In addition to the information contained in Exhibit 12-18, the following facts are known concerning this weaving segment:

PHF = 1.00 (all demands stated as flow rates);

Heavy vehicles = 0% trucks, 0% RVs (all demands in pc/h);

Driver population = regular commuters;

FFS = 75 mi/h;

c_{IFL} = 2,400 pc/h/ln (for FFS = 75 mi/h);

ID = 1 int/mi; and

Terrain = level.

Comments

As is the case in any weaving segment design, there are considerable constraints imposed. The problem states that the maximum length is 1,000 ft, no doubt limited by locational issues for the merge and diverge junctions. It is probably not worth investigating shorter lengths, and the maximum should be assumed for all trial designs. The simplest design merely connects entering lanes with exit lanes in a straightforward manner, producing a section of five lanes. A section with four lanes could be considered by merging two lanes into one at the entry gore and separating it into two again at the exit gore. In any event, the design is limited to a section of four or five lanes. No other widths would work without major additions to input and output legs. The configuration cannot be changed without adding a lane to at least one of the entry or exit legs. Thus, the initial trial will be at a length of 1,000 ft, with the five entry lanes connected directly to the five exit lanes, with no changes to the exit or entry leg designs. If this does not produce an acceptable operation, changes will be considered.

While the problem clearly states that all legs are freeways, no feasible configuration produces a two-sided weaving section. Thus, to fit within the one-sided analysis methodology, the right-side entry and exit legs will be classified as ramps in the computational analysis.

Step 1: Input Data – Trial 1

All input information is given in Exhibit 12-18 and in the accompanying Facts section for this example problem.

Step 2: Adjust Volume – Trial 1

All demands are already stated as flow rates in passenger cars per hour under equivalent ideal conditions. No further adjustments are needed. Critical demand values are as follows:

$$v_{FF} = 2{,}000 \text{ pc/h}$$

$$v_{FR} = 1{,}450 \text{ pc/h}$$

$$v_{RF} = 1{,}500 \text{ pc/h}$$

$$v_{RR} = 2{,}000 \text{ pc/h}$$

$$v_W = 1{,}500 + 1{,}450 = 2{,}950 \text{ pc/h}$$

$$v_{NW} = 2{,}000 + 2{,}000 = 4{,}000 \text{ pc/h}$$

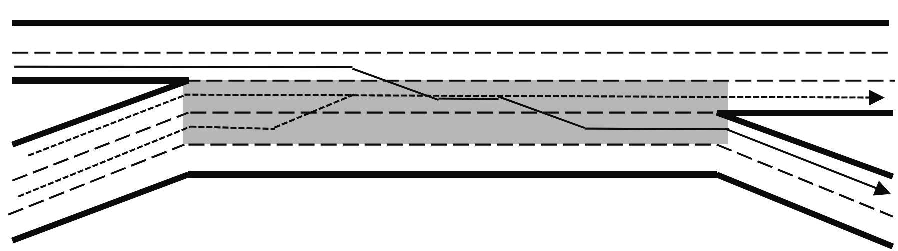
$$v = 2{,}950 + 4{,}000 = 6{,}950 \text{ pc/h}$$

$$VR = \frac{2{,}950}{6{,}950} = 0.424$$

Step 3: Determine Configuration Characteristics – Trial 1

Exhibit 12-19 illustrates the weaving segment formed under the assumed design discussed previously.

Exhibit 12-19
Trial Design 1
for Example Problem 4



The direct connection of entry and exit legs produces a weaving segment in which the ramp-to-freeway movement can be made without a lane change (LC_{RF} = 0). Freeway-to-ramp vehicles, however, must make two lane changes (LC_{FR} = 2). If the lane-changing pattern is considered, there are no lanes on the entering freeway leg from which a weaving maneuver can be made with one or no lane changes. Ramp drivers wishing to weave, however, can enter on either of the two left ramp lanes and weave with one or no lane changes. Thus, N_{WL} = 2.

By using Equation 12-2, LC_{MIN} is computed as

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

$$LC_{MIN} = (0 \times 1{,}500) + (2 \times 1{,}450) = 2{,}900 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length – Trial 1

The maximum length of a weaving segment for this configuration and demand scenario is estimated by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.424)^{1.6}\right] - \left[1{,}566 \times 2\right] = 6{,}950 \text{ ft} > 1{,}000 \text{ ft}$$

As the maximum length is much greater than the actual length of 1,000 ft, it is appropriate to analyze the segment by using this chapter's methodology.

Step 5: Determine Weaving Segment Capacity – Trial 1

The capacity of the weaving segment is controlled by one of two limiting factors: density reaches 43 pc/mi/ln or weaving demand reaches 2,400 pc/h for the configuration of Exhibit 12-19.

Capacity Limited by Density

The capacity limited by reaching a density of 43 pc/mi/ln is estimated by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765 L_s\right] + \left[119.8 N_{WL}\right]$$

$$c_{IWL} = 2{,}400 - \left[438.2(1+0.424)^{1.6}\right] + \left[0.0765 \times 1{,}000\right] + \left[119.8 \times 2\right]$$

$$c_{IWL} = 1{,}944 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 1{,}944 \times 5 \times 1 \times 1 = 9{,}721 \text{ pc/h}$$

Capacity Limited by Weaving Demand Flow

The capacity limited by the weaving demand flow is estimated by using Equation 12-7 and Equation 12-8:

$$c_{IW} = \frac{2{,}400}{VR} = \frac{2{,}400}{0.424} = 5{,}654 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} \times f_p = 5{,}654 \times 1 \times 1 = 5{,}654 \text{ pc/h}$$

In this case, the capacity of the segment is limited by the maximum weaving flow rate of 5,654 pc/h, which is smaller than the total demand flow rate of 6,950 pc/h. Thus, this section is expected to operate at LOS F. No further analysis is possible with this methodology.

Discussion – Trial 1

This section would be expected to fail under the proposed design. The critical feature appears to be the configuration. Note that the capacity is limited by the maximum weaving flows that can be sustained, not by a density expected to produce queuing. This is primarily due to the freeway-to-ramp flow, which must make two lane changes. This number can be reduced to one by adding one lane to the "ramp" at the exit gore area. Not only does this reduce the number of lane changes made by 1,450 freeway-to-ramp vehicles, but it also increases the value of N_W from 2 to 3. In turn, this effectively increases the segment's capacity

(as limited by weaving flow rate) to 3,500/VR = 3,500/0.424 = 8,255 pc/h, which is well in excess of the demand flow rate of 6,950 pc/h. Another analysis (Trial 2) will be conducted by using this approach.

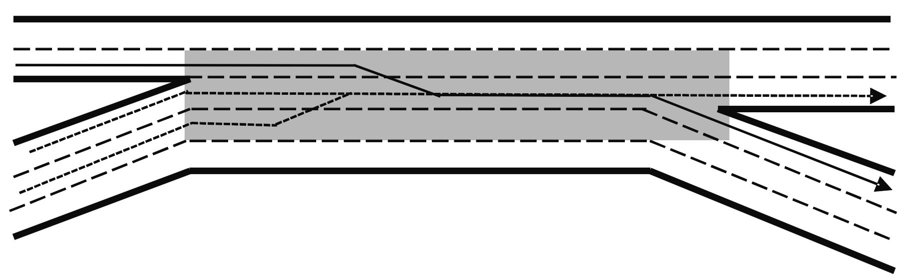
Steps 1 and 2: Input Data and Adjust Volume – Trial 2

Steps 1 and 2 are the same as for Trial 1. They are not repeated here. The new configuration affects the results beginning with Step 3.

Step 3: Determine Configuration Characteristics – Trial 2

Exhibit 12-20 illustrates the new configuration that will result from the changes discussed above. By adding a lane to the exit-ramp leg, the freeway-to-ramp movement can now be completed with only one lane change (LC_{FR} = 1). The value of LC_{RF} is not affected and remains 0. The right lane of the freeway-entry leg can also be used by freeway-to-ramp drivers to make a weaving maneuver with a single lane change, increasing N_{WL} to 3.

Exhibit 12-20
Trial Design 2
for Example Problem 4



Then

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

$$LC_{MIN} = (0 \times 1{,}500) + (1 \times 1{,}450) = 1{,}450 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length – Trial 2

The maximum length of a weaving segment for this configuration and demand scenario is estimated by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1 + VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1 + 0.424)^{1.6}\right] - \left[1{,}566 \times 3\right] = 5{,}391 \text{ ft} > 1{,}000 \text{ ft}$$

As the maximum length is much greater than the actual length of 1,000 ft, analyzing the segment by using this chapter's methodology is appropriate.

Step 5: Determine Weaving Segment Capacity – Trial 2

The capacity of the weaving segment is controlled by one of two limiting factors: density reaches 43 pc/mi/ln or weaving demand reaches 3,500 pc/h for the configuration of Exhibit 12-20.

Capacity Limited by Density

The capacity limited by reaching a density of 43 pc/mi/ln is estimated by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_s\right] + \left[119.8N_{WL}\right]$$

$$c_{IWL} = 2{,}400 - \left[438.2(1+0.424)^{1.6}\right] + \left[0.0765 \times 1{,}000\right] + \left[119.8 \times 3\right]$$

$$c_{IWL} = 2{,}064 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 2{,}064 \times 5 \times 1 \times 1 = 10{,}320 \text{ pc/h}$$

Capacity Limited by Weaving Demand Flow

The capacity limited by the weaving demand flow is estimated by using Equation 12-7 and Equation 12-8:

$$c_{IW} = \frac{3{,}500}{VR} = \frac{3{,}500}{0.424} = 8{,}255 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} \times f_p = 8{,}255 \times 1 \times 1 = 8{,}255 \text{ pc/h}$$

Once again, the capacity of the segment is limited by the maximum weaving flow rate: the difference is that now the capacity is 8,255 pc/h. This is larger than the total demand flow rate of 6,950 pc/h. Thus, this section is expected to operate without breakdown, and the analysis may continue.

Step 6: Determine Lane-Changing Rates – Trial 2

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}(N^2)(1+ID)^{0.8}\right]$$

$$LC_W = 1{,}450 + 0.39\left[(1{,}000 - 300)^{0.5} 5^2 (1+1)^{0.8}\right] = 1{,}899 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{1{,}000 \times 1 \times 4{,}000}{10{,}000} = 400 < 1{,}300$$

$$LC_{NW} = (0.206v_{NW}) + (0.542L_S) - (192.6N)$$

$$LC_{NW} = (0.206 \times 4{,}000) + (0.542 \times 1{,}000) - (192.6 \times 5) = 403 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}899 + 403 = 2{,}302 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles – Trial 2

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20.

$$W = 0.226\left(\frac{LC_{ALL}}{L_s}\right)^{0.789} = 0.226\left(\frac{2{,}302}{1{,}000}\right)^{0.789} = 0.436$$

Then

$$S_W = 15 + \left(\frac{FFS - 15}{1 + W}\right) = 15 + \left(\frac{75 - 15}{1 + 0.436}\right) = 56.8 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048\frac{v}{N}\right)$$

$$S_{NW} = 75 - (0.0072 \times 1{,}450) - \left(0.0048\frac{6{,}950}{5}\right) = 57.9 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{4{,}000 + 2{,}950}{\left(\frac{4{,}000}{57.9}\right) + \left(\frac{2{,}950}{56.8}\right)} = 57.4 \text{ mi/h}$$

Step 8: Determine the Level of Service – Trial 2

The average density in the weaving segment is estimated by using Equation 12-22:

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{6{,}950}{5}\right)}{57.4} = 24.2 \text{ pc/mi/ln}$$

From Exhibit 12-10, this density is within the stated boundaries of LOS C (20 to 28 pc/mi/ln). As the design target was LOS C, the second trial design is acceptable.

Discussion – Trial 2

The relatively small change in the configuration makes all the difference in this design. LOS C can be achieved by adding a lane to the right exit leg; without it, the section fails due to excessive weaving turbulence. If the extra lane is not needed on the departing freeway leg, it would be dropped somewhere downstream, perhaps as part of the next interchange. The extra lane would have to be carried for several thousand feet to be effective. An added lane generally will not be fully utilized by drivers if they are aware that it will be immediately dropped.

EXAMPLE PROBLEM 5: CONSTRUCTING A SERVICE VOLUME TABLE FOR A WEAVING SEGMENT

This example shows how a table of service flow rates or service volumes or both can be constructed for a weaving section with certain specified characteristics. The methodology of this chapter does not directly yield service

flow rates or service volumes, but they can be developed by using spreadsheets or more sophisticated computer programs.

The key issue is the definition of the threshold values for the various levels of service. For weaving sections on freeways, levels of service are defined as limiting densities as follows:

LOS	Maximum Density (pc/mi/ln)
A	10
B	20
C	28
D	35

By definition, the service flow rate at LOS E is the capacity of the weaving section, which may or may not be keyed to a density.

Before the construction of such a table is illustrated, several key definitions should be reviewed:

- *Service flow rate (under ideal conditions):* The maximum rate of flow under equivalent ideal conditions that can be sustained while maintaining the designated LOS (*SFI*, passenger cars per hour).
- *Service flow rate (under prevailing conditions):* The maximum rate of flow under prevailing conditions that can be sustained while maintaining the designated LOS (*SF*, vehicles per hour).
- *Service volume:* The maximum hourly volume under prevailing conditions that can be sustained while maintaining the designated LOS in the worst 15 min of the hour (*SV*, vehicles per hour).
- *Daily service volume:* The maximum AADT under prevailing conditions that can be sustained while maintaining the designated LOS in the worst 15 min of the peak hour (*DSV*, vehicles per day).

Note that flow rates are for a 15-min period, often a peak 15 min within the analysis hour, or the peak hour. These values are related as follows:

$$SF_i = SFI_i \times f_{HV} \times f_p$$

$$SV_i = SF_i \times PHF$$

$$DSV_i = \frac{SV_i}{KD}$$

This chapter's methodology estimates both the capacity and the density expected in a weaving segment of given geometric and demand characteristics. Conceptually, the approach to generating values of *SFI* is straightforward: for any given situation, keep increasing the input flow rates until the boundary density for the LOS is reached; the input flow rate is the *SFI* for that situation and LOS. This obviously involves many iterations. A spreadsheet can be programmed to do this, either semiautomatically with manual input of demands, or fully automatically, with the spreadsheet automatically generating solutions until a density match is found. The latter method is not very efficient and involves a typical spreadsheet program running for several hours. A program could, of course, be written to automate the entire process.

An Example

While all of the computations cannot be shown, demonstration results for a specific case can be illustrated. A service volume table is desired for a weaving section with the following characteristics:

- One-sided major weaving section
- Demand splits as follows:
  - v_{FF} = 65% of v
  - v_{RF} = 15% of v
  - v_{FR} = 12% of v
  - v_{RR} = 8% of v
- Trucks = 10%, RVs = 0%
- Level terrain
- PHF = 0.93
- f_p = 1.00
- *ID* = 1 int/mi
- FFS = 65 mi/h

For these characteristics, a service volume table can be constructed for a range of lengths and widths and for configurations in which N_W is 2 and 3. For illustrative purposes, lengths of 500, 1,000, 1,500, 2,000, and 2,500 ft and widths of three, four, or five lanes will be used. In a major weaving section, one weaving flow does not have to make a lane change. For the purposes of this example, it is assumed that the ramp-to-freeway movement has this characteristic. The freeway-to-ramp movement would require one or two lane changes, on the basis of the value of N_{WL}.

First Computations

Initial computations will be aimed at establishing values of *SFI* for the situations described. A spreadsheet will be constructed in which the first column is the flow rate to be tested (in passenger cars per hour under ideal conditions), and the last column produces a density. Each line will be iterated (manually in this case) until each threshold density value is reached. Intermediate columns will be programmed to produce the intermediate results needed to get to this result. Because maximum length and capacity are decided at intermediate points, the applicable results will be manually entered before continuing. Such a procedure is less difficult than it seems once the basic computations are programmed. Manual iteration using the input flow rate is very efficient, as the operator will observe how fast the results are converging to the desired threshold and will change the inputs accordingly.

The results of a first computation are shown in Exhibit 12-21. They represent service flow rates under ideal conditions, *SFI*. Consistent with the HCM's results presentation guidelines (Chapter 7, Interpreting HCM and Alternative Tool Results), all hourly service flow rates and volumes in the following exhibits have

been rounded down to the nearest 100 passenger cars or vehicles for presentation.

Exhibit 12-21
Service Flow Rates Under Ideal Conditions (*SFI*) for Example Problem 5 (pc/h)

LOS	Length of Weaving Section (ft)									
	500	1,000	1,500	2,000	2,500	500	1,000	1,500	2,000	2,500
	$N = 3$; $N_{WL} = 2$					$N = 3$; $N_{WL} = 3$				
A	1,700	1,700	1,700	1,700	1,700	1,800	1,800	1,800	1,800	1,800
B	3,200	3,200	3,200	3,200	3,200	3,300	3,300	3,400	3,400	3,400
C	4,200	4,200	4,300	4,300	4,300	4,400	4,500	4,500	4,500	4,500
D	5,000	5,100	5,100	5,100	5,100	5,300	5,400	5,400	5,500	5,500
E	5,900	6,000	6,100	6,300	6,400	6,300	6,400	6,500	6,600	6,700
	$N = 4$; $N_{WL} = 2$					$N = 4$; $N_{WL} = 3$				
A	2,200	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300
B	4,100	4,200	4,200	4,200	4,200	4,300	4,400	4,400	4,400	4,400
C	5,400	5,500	5,500	5,500	5,600	5,800	5,900	5,900	5,900	5,900
D	6,300	6,500	6,500	6,600	6,600	6,900	7,000	7,100	7,100	7,100
E	7,900	8,000	8,200	8,400	8,500	8,400	8,500	8,700	8,800	9,000
	$N = 5$; $N_{WL} = 2$					$N = 5$; $N_{WL} = 3$				
A	2,800	2,800	2,800	2,800	2,800	2,900	2,900	2,900	2,900	2,900
B	5,000	5,100	5,100	5,100	5,100	5,400	5,400	5,400	5,500	5,500
C	6,500	6,600	6,700	6,700	6,700	7,100	7,200	7,200	7,300	7,300
D	7,600	7,800	7,900	7,900	7,900	8,400	8,600	8,700	8,700	8,700
E	8,800	8,800	8,800	8,800	8,800	10,500	10,700	10,900	11,100	11,200

Exhibit 12-22 shows service flow rates under prevailing conditions, *SF*. Each value in Exhibit 12-21 (before rounding) is multiplied by

$$f_{HV} = \frac{1}{1+0.10(1.5-1)} = 0.952$$

$$f_p = 1.00$$

Exhibit 12-22
Service Flow Rates Under Prevailing Conditions (*SF*) for Example Problem 5 (veh/h)

LOS	Length of Weaving Section (ft)									
	500	1,000	1,500	2,000	2,500	500	1,000	1,500	2,000	2,500
	$N = 3$; $N_{WL} = 2$					$N = 3$; $N_{WL} = 3$				
A	1,600	1,600	1,600	1,600	1,600	1,700	1,700	1,700	1,700	1,700
B	3,000	3,000	3,100	3,100	3,100	3,100	3,200	3,200	3,200	3,200
C	4,000	4,000	4,100	4,100	4,100	4,200	4,300	4,300	4,300	4,300
D	4,700	4,800	4,900	4,900	4,900	5,100	5,100	5,200	5,200	5,200
E	5,600	5,700	5,800	5,900	6,100	6,000	6,100	6,200	6,200	6,400
	$N = 4$; $N_{WL} = 2$					$N = 4$; $N_{WL} = 3$				
A	2,100	2,100	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200
B	3,900	4,000	4,000	4,000	4,000	4,100	4,200	4,200	4,200	4,200
C	5,100	5,200	5,200	5,300	5,300	5,500	5,600	5,600	5,600	5,600
D	5,900	6,200	6,200	6,300	6,300	6,600	6,700	6,700	6,800	6,800
E	7,500	7,700	7,800	7,900	8,100	8,000	8,100	8,200	8,400	8,500
	$N = 5$; $N_{WL} = 2$					$N = 5$; $N_{WL} = 3$				
A	2,600	2,700	2,700	2,700	2,700	2,700	2,700	2,800	2,800	2,800
B	4,700	4,800	4,900	4,900	4,900	5,100	5,100	5,200	5,200	5,200
C	6,200	6,300	6,300	6,400	6,400	6,700	6,800	6,900	6,900	6,900
D	7,300	7,400	7,500	7,500	7,500	8,000	8,200	8,200	8,300	8,300
E	8,400	8,400	8,400	8,400	8,400	10,000	10,200	10,300	10,500	10,700

Exhibit 12-23 shows service volumes, *SV*. Each value in Exhibit 12-22 (before rounding) is multiplied by a PHF of 0.93.

Exhibit 12-23
Service Volumes Under Prevailing Conditions (*SV*) for Example Problem 5 (veh/h)

LOS	Length of Weaving Section (ft) 500	1,000	1,500	2,000	2,500	500	1,000	1,500	2,000	2,500
	***N* = 3; N_{WL} = 2**					***N* = 3; N_{WL} = 3**				
A	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500
B	2,800	2,800	2,800	2,800	2,900	2,900	2,900	3,000	3,000	3,000
C	3,700	3,700	3,800	3,800	3,800	3,900	4,000	4,000	4,000	4,000
D	4,400	4,500	4,500	4,500	4,500	4,700	4,800	4,800	4,800	4,800
E	5,200	5,300	5,400	5,500	5,600	5,500	5,600	5,700	5,800	5,900
	***N* = 4; N_{WL} = 2**					***N* = 4; N_{WL} = 3**				
A	2,000	2,000	2,000	2,000	2,000	2,000	2,100	2,100	2,100	2,100
B	3,600	3,700	3,700	3,700	3,700	3,800	3,900	3,900	3,900	3,900
C	4,700	4,800	4,900	4,900	4,900	5,100	5,200	5,200	5,200	5,200
D	5,500	5,700	5,800	5,800	5,800	6,100	6,200	6,300	6,300	6,300
E	7,000	7,100	7,300	7,400	7,500	7,400	7,500	7,700	7,800	7,900
	***N* = 5; N_{WL} = 2**					***N* = 5; N_{WL} = 3**				
A	2,400	2,500	2,500	2,500	2,500	2,500	2,500	2,600	2,600	2,600
B	4,400	4,500	4,500	4,500	4,500	4,700	4,800	4,800	4,800	4,800
C	5,700	5,800	5,900	5,900	5,900	6,200	6,400	6,400	6,400	6,400
D	6,700	6,900	7,000	7,000	7,000	7,500	7,600	7,700	7,700	7,700
E	7,800	7,800	7,800	7,800	7,800	9,300	9,400	9,600	9,800	9,900

Exhibit 12-24 shows daily service volumes, *DSV*. An illustrative K-factor of 0.08 (typical of a large urban area) and an illustrative D-factor of 0.55 (typical of an urban route without strong peaking by direction) are used. Each (nonrounded) value used to generate Exhibit 12-23 was divided by both of these numbers.

Exhibit 12-24
Daily Service Volumes Under Prevailing Conditions (*DSV*) for Example Problem 5 (veh/day)

LOS	Length of Weaving Section (ft) 500	1,000	1,500	2,000	2,500	500	1,000	1,500	2,000	2,500
	***N* = 3; N_{WL} = 2**					***N* = 3; N_{WL} = 3**				
A	35,200	35,200	35,400	35,500	35,600	36,200	36,300	36,300	36,300	36,300
B	64,300	65,300	65,500	65,700	66,100	67,600	68,000	68,400	68,400	68,400
C	84,700	86,100	86,700	87,200	87,500	89,700	90,900	91,500	91,700	91,900
D	100,800	102,800	103,600	104,000	104,400	107,800	109,600	110,200	110,600	110,800
E	119,800	122,100	124,400	126,700	129,100	127,000	129,400	131,600	132,800	136,300
	***N* = 4; N_{WL} = 2**					***N* = 4; N_{WL} = 3**				
A	45,800	46,200	46,600	46,600	46,600	47,600	47,800	47,800	47,900	47,900
B	83,300	84,700	85,100	85,500	85,700	88,300	89,300	89,500	89,700	89,900
C	108,600	110,800	111,600	112,200	112,600	117,100	118,700	119,500	120,100	120,300
D	126,700	131,300	132,400	133,200	133,600	140,000	142,400	143,600	144,000	144,400
E	159,800	162,800	165,900	169,000	172,100	169,400	172,500	175,400	178,600	181,700
	***N* = 5; N_{WL} = 2**					***N* = 5; N_{WL} = 3**				
A	56,300	57,100	57,300	57,500	57,500	58,700	58,900	59,300	59,400	59,400
B	101,400	103,000	103,600	104,200	104,400	108,600	109,600	110,000	110,600	110,800
C	131,300	133,800	135,000	135,800	136,200	142,800	145,400	146,200	146,800	147,400
D	154,500	157,700	159,100	159,900	160,300	170,600	173,600	175,000	175,800	175,800
E	178,800	178,800	178,800	178,800	178,800	211,800	215,600	219,500	223,300	227,200

6. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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CHAPTER 13
FREEWAY MERGE AND DIVERGE SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Freeway merge and diverge segments occur primarily at on-ramp and off-ramp junctions with the freeway mainline. They can also occur at major merge or diverge points where mainline roadways join or separate.

Freeway merge and diverge segments include ramp junctions and points where mainline roadways join or separate.

A ramp is a dedicated roadway providing a connection between two highway facilities. On freeways, all movements onto and off of the freeway are made at ramp junctions—designed to permit relatively high-speed merging and diverging maneuvers while limiting the disruption to the main traffic stream. Some ramps on freeways connect to collector–distributor (C-D) roadways, which in turn provide a junction with the freeway mainline. Ramps may appear on multilane highways, two-lane highways, arterials, and urban streets, but such facilities may also use signalized and unsignalized intersections at such junctions.

The procedures in **Chapter 13, Freeway Merge and Diverge Segments,** focus on ramp–freeway junctions, but guidance is also provided to allow approximate use of such procedures on multilane highways and on C-D roadways.

This chapter provides guidance for using the procedures on multilane highways and C-D roadways.

RAMP COMPONENTS

A ramp consists of three elements: the ramp roadway and two junctions. Junctions vary greatly in design and control features but generally fit into one of these categories:

- Ramp–freeway junctions (or a junction with a C-D roadway or multilane highway segment), or
- Ramp–street junctions.

When a ramp connects one freeway to another, the ramp consists of two ramp–freeway junctions and the ramp roadway. When a ramp connects a freeway to a surface facility, it generally consists of a ramp–freeway junction, the ramp roadway, and a ramp–street junction. When a ramp connection to a surface facility (such as a multilane highway) or a C-D roadway is designed for high-speed merging or diverging without control, it may be classified as a ramp–freeway junction for the purpose of analysis.

Ramps to multilane highways and C-D roadways that are designed for high-speed merging or diverging may be classified as ramp–freeway junctions for analysis purposes.

Ramp–street junctions may be uncontrolled, STOP-controlled, YIELD-controlled, or signalized. Analysis of ramp–street junctions is not detailed in this chapter; rather, it is discussed in Chapter 22, Interchange Ramp Terminals. Note, however, that an off-ramp–street junction, particularly if signalized, can result in queuing on the ramp roadway that can influence operations at the ramp–freeway junction and even mainline freeway conditions. Mainline operations can also be affected by platoon entries created by ramp–street intersection control.

See Chapter 22 for a discussion of ramp–street junctions.

Ramp queuing from a junction of an off-ramp and street can influence the operations of the ramp–freeway junction and the upstream freeway.

The geometric characteristics of ramp–freeway junctions vary. The length and type (parallel, taper) of acceleration or deceleration lane(s), the free-flow speed (FFS) of both the ramp and the freeway in the vicinity of the ramp, proximity of other ramps, and other elements all affect merging and diverging operations.

CLASSIFICATION OF RAMPS

Ramps and ramp–freeway junctions may occur in a wide variety of configurations. Some of the key characteristics of ramps and ramp junctions are summarized below:

- Ramp–freeway junctions that accommodate merging maneuvers are classified as *on-ramps.* Those that accommodate diverging maneuvers are classified as *off-ramps.* Where the junctions accommodate the merging of two major facilities, they are classified as *major merge* junctions. Where they accommodate the divergence of two major roadways, they are classified as *major diverge* junctions.
- The majority of ramps are right-hand ramps. Some, however, join with the left lane(s) of the freeway and are classified as left-hand ramps.
- Ramp roadways may have one or two lanes. At on-ramp freeway junctions, most two-lane ramp roadways merge into a single lane before merging with the freeway. In this case, the junction is classified as a one-lane ramp–freeway junction on the basis of the methodology of this chapter. In other cases, a two-lane ramp–freeway merge exists, and a special analysis model is used (see this chapter's Special Cases section).
- For two-lane off-ramps, a single lane may exist at the ramp–freeway diverge, with the roadway widening to two lanes after the diverge. As with on-ramps, such cases are classified as one-lane ramp–freeway junctions on the basis of this chapter's methodology. Two-lane off-ramp roadways, however, often have two lanes at the diverge point as well. These are treated by using a special model (see this chapter's Special Cases section).
- Ramp–freeway merge and diverge operations are affected by the size of the freeway segment (in one direction).
- Ramp–freeway merge and diverge operations may be affected by the proximity of adjacent ramps and the flow rates on those ramps.

Left-hand ramps are considered as special cases later in this chapter.

Merge and diverge segments with two lanes at the point of merge or diverge are considered as special cases later in this chapter.

The number of combinations of these characteristics that can occur is very large. For any analysis, all of these (and other) characteristics must be specified if meaningful results are to be obtained.

RAMP AND RAMP JUNCTION ANALYSIS BOUNDARIES

Ramps and ramp junctions do not operate independently of the roadways they connect. Thus, operating conditions on the main roadways can affect operations on the ramp and ramp junctions, and vice versa. In particular, a breakdown [Level of Service (LOS) F] at a ramp–freeway junction may have serious effects on the freeway upstream or downstream of the junction. These effects can influence freeway operations for miles in the worst cases.

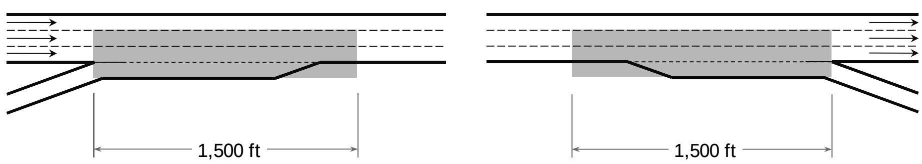
With undersaturated conditions, the operational impacts of ramp–freeway junctions occur within a 1,500-ft-long influence area.

For most stable operations, however, studies (*1*) have shown that the operational impacts of ramp–freeway junctions are more localized. Thus, the methodology presented in this chapter predicts the operating characteristics within a defined ramp influence area. For right-hand on-ramps, the ramp influence area includes the acceleration lane(s) and Lanes 1 and 2 of the freeway

mainline (rightmost and second rightmost) for a distance of 1,500 ft downstream of the merge point. For right-hand off-ramps, the ramp influence area includes the deceleration lane(s) and Lanes 1 and 2 of the freeway for a distance of 1,500 ft upstream of the diverge point. Exhibit 13-1 illustrates the definition of ramp influence areas. For left-hand ramps, the two leftmost lanes of the freeway are affected.

The influence area includes the acceleration/deceleration lane and the right two lanes of the freeway (left two lanes for left-hand ramps).

Exhibit 13-1
Ramp Influence Areas Illustrated



(a) Merge Influence Area (b) Diverge Influence Area

RAMP–FREEWAY JUNCTION OPERATIONAL CONDITIONS

Ramp–freeway junctions create turbulence in the merging or diverging traffic stream. In general, the turbulence is the result of high lane-changing rates.

The action of individual merging vehicles entering the Lane 1 traffic stream creates turbulence in the vicinity of the ramp. Approaching freeway vehicles move toward the left to avoid the turbulence. Thus, the ramp influence area experiences a higher rate of lane-changing than is normally present on ramp-free portions of freeway.

Ramp influence areas experience higher rates of lane-changing than normally occur in basic freeway segments.

At off-ramps, the basic maneuver is a diverge—a single traffic stream separating into two streams. Exiting vehicles must occupy the lane(s) adjacent to the off-ramp (Lane 1 for a single-lane right-hand off-ramp). Thus, as the off-ramp is approached, vehicles leaving the freeway must move to the right. This causes other freeway vehicles to redistribute as they move left to avoid the turbulence of the immediate diverge area. Again, the ramp influence area has a higher rate of lane-changing than is normally present on ramp-free portions of freeway.

Vehicle interactions are dynamic in ramp influence areas. Approaching freeway through vehicles will move left as long as there is capacity to do so. Whereas the intensity of ramp flow influences the behavior of through freeway vehicles, general freeway congestion can also act to limit ramp flow, causing diversion to other interchanges or routes.

Exhibit 13-1 and the accompanying discussion relate to single-lane right-hand ramps. For two-lane right-hand ramps, the characteristics are basically the same, except that two acceleration or deceleration lanes may be present. For left-hand ramps, merging and diverging obviously take place on the left side of the freeway. This chapter's methodology is based on right-hand ramps, but modifications allowing the adaptation of the methodology to consider left-hand ramps are presented in the Special Cases section of this chapter.

BASE CONDITIONS

The base conditions for the methodology presented in this chapter are the same as for other types of freeway segments:

Base conditions for merge and diverge segments are the same as for other types of freeway segments.

- No heavy vehicles,
- 12-ft lanes,
- Adequate lateral clearances (≥6 ft), and
- Road users familiar with the facility (i.e., $f_p = 1.00$).

LOS CRITERIA FOR MERGE AND DIVERGE SEGMENTS

LOS A–E is defined in terms of density; LOS F exists when demand exceeds capacity.

Merge/diverge segment LOS is defined in terms of density for all cases of stable operation (LOS A–E). LOS F exists when the freeway demand exceeds the capacity of the upstream (diverges) or downstream (merges) freeway segment, or where the off-ramp demand exceeds the off-ramp capacity.

At LOS A, unrestricted operations exist, and the density is low enough to permit smooth merging or diverging with very little turbulence in the traffic stream. At LOS B, merging and diverging maneuvers become noticeable to through drivers, and minimal turbulence occurs. At LOS C, speed within the ramp influence area begins to decline as turbulence levels become much more noticeable. Both ramp and freeway vehicles begin to adjust their speeds to accomplish smooth transitions. At LOS D, turbulence levels in the influence area become intrusive, and virtually all vehicles slow to accommodate merging or diverging maneuvers. Some ramp queues may form at heavily used on-ramps, but freeway operation remains stable. LOS E represents operating conditions approaching or at capacity. Small changes in demand or disruptions within the traffic stream can cause both ramp and freeway queues to form.

LOS F defines operating conditions within queues that form on both the ramp and the freeway mainline when capacity is exceeded by demand. For on-ramps, LOS F exists when the total demand flow rate from the upstream freeway segment and the on-ramp exceeds the capacity of the downstream freeway segment. For off-ramps, LOS F exists when the total demand flow rate on the approaching upstream freeway segment exceeds the capacity of the upstream freeway segment. LOS F also occurs when the off-ramp demand exceeds the capacity of the off-ramp.

Exhibit 13-2 summarizes the LOS criteria for freeway merge and diverge segments. These criteria apply to all ramp–freeway junctions and may also be applied to major merges and diverges; high-speed, uncontrolled merge or diverge ramps on multilane highway sections; and merges and diverges on freeway C-D roadways. LOS is not defined for ramp roadways, while the LOS of a ramp–street junction is defined in Chapter 22, Interchange Ramp Terminals.

Exhibit 13-2
LOS Criteria for Freeway Merge and Diverge Segments

LOS	Density (pc/mi/ln)	Comments
A	≤10	Unrestricted operations
B	>10–20	Merging and diverging maneuvers noticeable to drivers
C	>20–28	Influence area speeds begin to decline
D	>28–35	Influence area turbulence becomes intrusive
E	>35	Turbulence felt by virtually all drivers
F	Demand exceeds capacity	Ramp and freeway queues form

REQUIRED INPUT DATA

The analysis of a ramp–freeway junction requires details concerning the junction under analysis and adjacent upstream and downstream ramps, in addition to the data required for a typical freeway analysis.

Data Describing the Freeway

The following information concerning the freeway mainline is needed to conduct an analysis:

1. FFS: 55–75 mi/h;
2. Number of mainline freeway lanes: 2–5;
3. Terrain: level, rolling, or mountainous; or percent grade and length;
4. Heavy vehicle presence: percent trucks and buses, percent recreational vehicles (RVs);
5. Demand flow rate immediately upstream of the ramp–freeway junction;
6. Peak hour factor: up to 1.00; and
7. Driver population factor: 0.85–1.00.

FFS is best measured in the field but can be estimated by using the methodology for basic freeway segments or multilane highways, as applicable.

The freeway FFS is best measured in the field. If a field measurement is not available, one may be estimated by using the methodology for basic freeway segments presented in Chapter 11, Basic Freeway Segments. To use this methodology, information on lane widths, lateral clearances, number of lanes, and total ramp density is required. If the ramp junction is located on a multilane highway or C-D roadway, the FFS range is somewhat lower (45–60 mi/h) and can be estimated by using the methodology in Chapter 14, Multilane Highways, if no field measurements are available. The methodology can be applied to facilities with any FFS. Its use with multilane highways or C-D roadways must be considered approximate, however, since it was not calibrated with data from these types of facilities.

Where the ramp–freeway junction is on a specific grade, the length of the grade is measured from its beginning to the point of the ramp junction.

The driver population factor is generally 1.00, unless the demand consists primarily of drivers who are not regular users of the facility. In such cases, an appropriate value should be based on field observations at the location under study or at similar nearby locations.

Data Describing the Ramp–Freeway Junction

The following information concerning the ramp–freeway junction is needed to conduct an analysis:

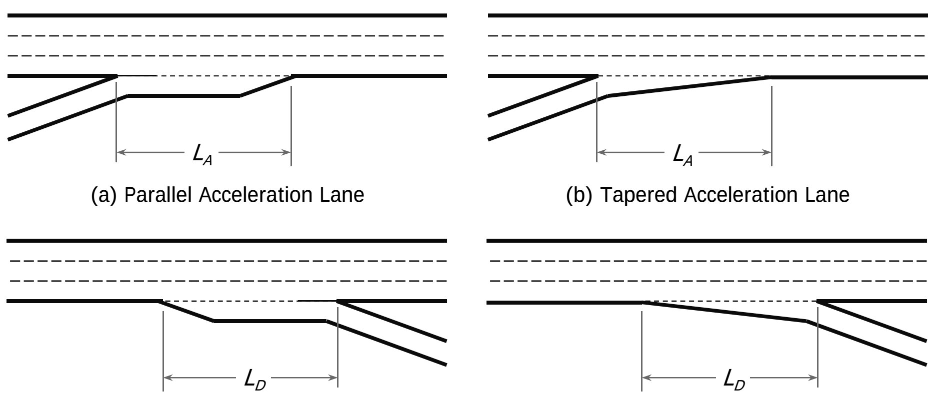
1. Type of ramp: on-ramp, off-ramp, major merge, major diverge;
2. Side of junction: right-hand, left-hand;
3. Number of lanes on ramp roadway: 1 lane, 2 lanes;
4. Number of ramp lanes at ramp–freeway junction: 1 lane, 2 lanes;
5. Length of acceleration/deceleration lane(s);
6. FFS of the ramp roadway: 20–50 mi/h;

7. Ramp terrain: level, rolling, or mountainous; or percent grade, length;
8. Demand flow rate on ramp;
9. Heavy vehicle presence: percent trucks and buses, percent RVs;
10. Peak hour factor: up to 1.00;
11. Driver population factor: 0.85–1.00; and
12. For adjacent upstream or downstream ramps:
    a. Upstream or downstream distance to the merge/diverge under study,
    b. Demand flow rate on the upstream or downstream ramp, and
    c. Peak hour factor and heavy vehicle percentages for the upstream or downstream ramp.

The length of the acceleration or deceleration lane includes the tapered portion of the ramp.

The length of the acceleration or deceleration lane includes the tapered portion of the ramp. Exhibit 13-3 illustrates lengths for both parallel and tapered ramp designs.

Exhibit 13-3
Measuring the Length of Acceleration and Deceleration Lanes



(a) Parallel Acceleration Lane (b) Tapered Acceleration Lane
(c) Parallel Deceleration Lane (d) Tapered Deceleration Lane

Source: *Traffic Engineering*, 3rd edition (*2*).

Length of Analysis Period

The analysis period for any freeway analysis, including ramp junctions, is generally the peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

2. METHODOLOGY

SCOPE OF THE METHODOLOGY

This chapter focuses on the operation of ramp–freeway junctions. The procedures may be applied in an approximate manner to completely uncontrolled ramp terminals on other types of facilities, such as multilane highways, two-lane highways, and freeway C-D roadways that are part of interchanges.

This chapter's procedures can be used to identify likely congestion at ramp–freeway junctions (LOS F) and to analyze undersaturated operations (LOS A–E) at ramp–freeway junctions. Chapter 10, Freeway Facilities, provides procedures for a more detailed analysis of oversaturated flow and congested conditions along a freeway section, including weaving, merge and diverge, and basic freeway segments.

The procedures in this chapter result primarily from studies conducted under National Cooperative Highway Research Program Project 3-37 (*1*, *2*). Some special applications resulted from adaptations of procedures developed in the 1970s (*3*). American Association of State Highway and Transportation Officials policies (*4*) contain additional material on the geometric design and design criteria for ramps.

LIMITATIONS OF THE METHODOLOGY

The methodology in this chapter does not take into account, nor is it applicable to (without modification by the analyst), cases involving

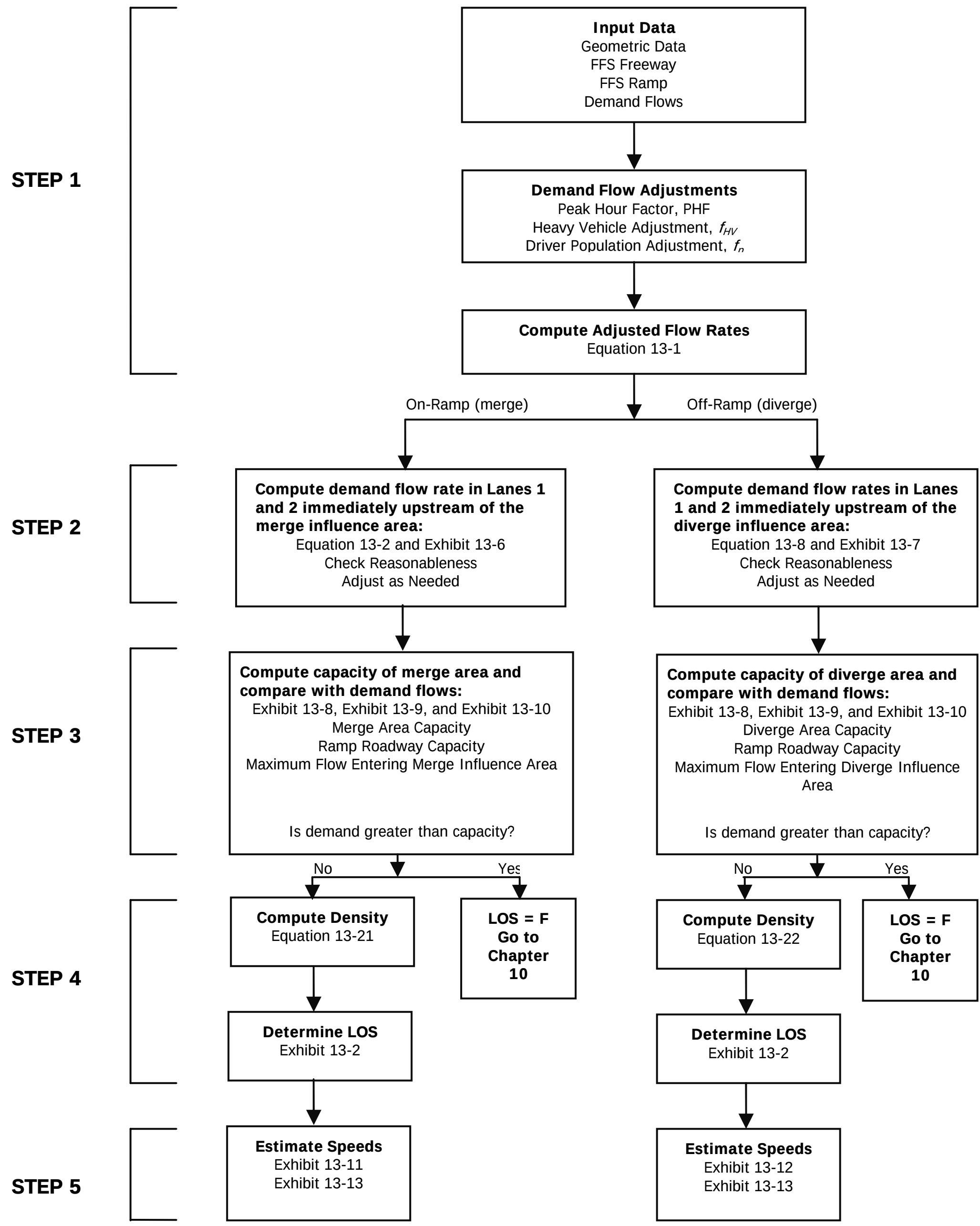
- Special lanes, such as high-occupancy vehicle (HOV) lanes, as ramp entry lanes;
- Ramp metering; or
- Intelligent transportation system features.

The methodology does not explicitly take into account posted speed limits or level of police enforcement. In some cases, low speed limits and strict enforcement could result in lower speeds and higher densities than those anticipated by this methodology.

OVERVIEW

Exhibit 13-4 illustrates the computational methodology applied to the analysis of ramp–freeway junctions. The analysis is generally entered with known geometric and demand factors. The primary outputs of the analysis are LOS and capacity. The methodology estimates the density and speed in the ramp influence area.

Exhibit 13-4
Flowchart for Analysis of Ramp–Freeway Junctions



As previously discussed, the methodology focuses on modeling the operating conditions within the ramp influence area, as defined in Exhibit 13-1. Because the ramp influence area includes only Lanes 1 and 2 of the freeway, an important part of the methodology involves predicting the number of approaching freeway vehicles that remain in these lanes immediately upstream of the ramp–freeway junction. While operations in other freeway lanes may be affected by merging and diverging maneuvers, particularly under heavy flow, the defined influence area experiences most of the operational impacts across all levels of service (except LOS F). At breakdown, queues and operational impacts may extend well beyond the defined influence area. Exhibit 13-5 illustrates key variables involved in the methodology.

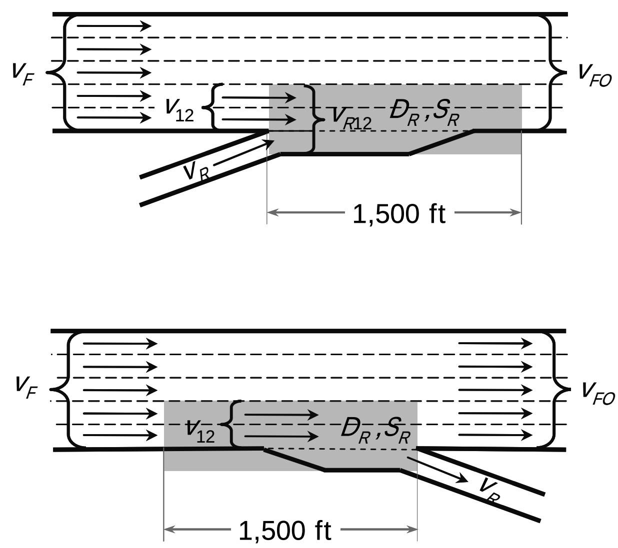


Exhibit 13-5
Key Ramp Junction Variables

The variables illustrated in Exhibit 13-5 are defined as follows:

v_F = flow rate on freeway immediately upstream of the ramp influence area under study (pc/h),

v_{12} = flow rate in freeway Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h),

v_{FO} = flow rate on the freeway immediately downstream of the merge or diverge area (pc/h),

v_R = flow rate on the on-ramp or off-ramp (pc/h),

v_{R12} = sum of the flow rates in Lanes 1 and 2 and the ramp flow rate (on-ramps only) (pc/h),

D_R = density in the ramp influence area (pc/mi/ln), and

S_R = average speed in the ramp influence area (mi/h).

The computational process illustrated in Exhibit 13-4 may be broken into five primary steps:

1. Specifying input variables and converting demand volumes to demand flow rates in passenger cars per hour under equivalent base conditions;
2. Estimating the flow remaining in Lanes 1 and 2 of the freeway immediately upstream of the merge or diverge influence area;
3. Estimating the capacity of the merge or diverge area and comparing the capacity with the converted demand flow rates;
4. For stable operations (i.e., demand is less than or equal to capacity), estimating the density within the ramp influence area and determining the expected LOS; and
5. When desired, estimating the average speed of vehicles within the ramp influence area.

Each step is discussed in detail in the sections that follow.

COMPUTATIONAL STEPS

The methodology was calibrated for one-lane, right-side ramp–freeway junctions. Other situations are addressed in the Special Cases section.

The methodology described in this section was calibrated for one-lane, right-side ramp–freeway junctions. All other cases—two-lane ramp junctions, left-side ramps, and major merge and diverge configurations—are analyzed with the modified procedures detailed in the Special Cases section.

Step 1: Specify Inputs and Convert Demand Volumes to Demand Flow Rates

All geometric and traffic variables for the ramp–freeway junction should be specified as inputs to the methodology, as discussed previously. Flow rates on the approaching freeway, on the ramp, and on any existing upstream or downstream adjacent ramps must be converted from hourly volumes (in vehicles per hour) to peak 15-min flow rates (in passenger cars per hour) under equivalent ideal conditions:

Equation 13-1

$$v_i = \frac{V_i}{PHF \times f_{HV} \times f_p}$$

where

v_i = demand flow rate for movement i (pc/h),

V_i = demand volume for movement i (veh/h),

PHF = peak hour factor,

f_{HV} = adjustment factor for heavy vehicle presence, and

f_p = adjustment factor for driver population.

If demand data or forecasts are already stated as 15-min flow rates, PHF is set at 1.00. Adjustment factors are the same as those used in Chapter 11, Basic Freeway Segments. These can also be used when the primary facility is a multilane highway or a C-D roadway in a freeway interchange.

Step 2: Estimate the Approaching Flow Rate in Lanes 1 and 2 of the Freeway Immediately Upstream of the Ramp Influence Area

Because the ramp influence area includes Lanes 1 and 2 of the freeway (for a right-hand ramp), a critical step in the analysis is estimating the total flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area.

The distribution of freeway vehicles approaching a ramp influence area is affected by a number of variables:

- Total freeway flow approaching the ramp influence area v_F (pc/h),
- Total on- or off-ramp flow v_R (pc/h),
- Total length of the acceleration lane L_A or deceleration lane L_D (ft), and
- FFS of the ramp at the junction point S_{FR} (mi/h).

The lane distribution of approaching freeway vehicles may also be affected by adjacent upstream or downstream ramps. Nearby ramps can influence lane distribution as drivers execute lane changes to position themselves for ramp movements at adjacent ramps. An on-ramp, for example, located only a few

hundred feet upstream of a subject ramp may result in additional vehicles in Lanes 1 and 2 at the subject ramp. A downstream off-ramp near a subject ramp may contain additional vehicles in Lanes 1 and 2 destined for the downstream ramp.

Theoretically, the influence of adjacent upstream and downstream ramps does not depend on the size of the freeway. In practical terms, however, this methodology only accounts for such influences on six-lane freeways (three lanes in one direction). On four-lane freeways (two lanes in one direction), the determination of v_{12} is trivial: since only Lanes 1 and 2 exist, all approaching freeway vehicles are, by definition, in Lanes 1 and 2 regardless of the proximity of adjacent ramps. On eight-lane (four lanes in one direction) or larger freeways, the data are insufficient to determine the impact of adjacent ramps on lane distribution. In addition, two-lane ramps are never included as “adjacent” ramps under these procedures.

For six-lane freeways, the methodology includes a process for determining whether adjacent upstream and downstream ramps are close enough to influence lane distribution at a subject ramp junction. When such ramps are close enough, the following additional variables may be involved:

- Flow rate on the adjacent upstream ramp v_U (pc/h),
- Distance between the subject ramp junction and the adjacent upstream ramp junction L_{UP} (ft),
- Flow rate on the adjacent downstream ramp v_D (pc/h), and
- Distance between the subject ramp junction and the adjacent downstream ramp junction L_{DOWN} (ft).

The distance to adjacent ramps is measured between the points at which the left edge of the leftmost ramp lane meets the right-lane edge of the freeway.

In practical terms, the influence of adjacent ramps rarely extends more than approximately 8,000 ft. Nevertheless, whether an adjacent ramp on a six-lane freeway has influence should be determined by using the algorithms specified in this methodology.

Of all these variables, the total approaching freeway flow has the greatest impact on flow in Lanes 1 and 2. The models are structured to account for this phenomenon without distorting other relationships. Longer acceleration and deceleration lanes lessen turbulence as ramp vehicles enter or leave the freeway. This leads to lower densities and higher speeds in the ramp influence area. When the ramp has a higher FFS, vehicles can enter and leave the freeway at higher speeds, and approaching freeway vehicles tend to move left to avoid the possibility of high-speed turbulence. This produces greater presegregation and smoother flow across all freeway lanes.

While the models are similarly structured, there are distinct differences between the lane distribution impacts of on-ramps and off-ramps. Separate models are presented for each case in the sections that follow.

Estimating Flow in Lanes 1 and 2 for On-Ramps (Merge Areas)

The general model for on-ramps specifies that flow in Lanes 1 and 2 immediately upstream of the merge influence area is simply a proportion of the approaching freeway flow, as shown in Equation 13-2:

Equation 13-2

$$v_{12} = v_F \times P_{FM}$$

where

v_{12} = flow rate in Lanes 1 and 2 (pc/h),

v_F = total flow rate on freeway immediately upstream of the on-ramp (merge) influence area (pc/h), and

P_{FM} = proportion of freeway vehicles remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area.

Exhibit 13-6 shows the algorithms used to determine P_{FM} for on-ramps or merge areas. All variables in Exhibit 13-6 are as previously defined.

Three equations are provided for six-lane freeways. Equation 13-3 is the base case for isolated ramps and for cases in which adjacent ramps are not found to influence merging operations. Equation 13-4 addresses cases with an upstream adjacent off-ramp, while Equation 13-5 addresses cases with a downstream adjacent off-ramp. Adjacent on-ramps (either upstream or downstream) have not been found to have a statistically significant impact on operations and are therefore ignored; Equation 13-3 is applied in such cases.

Adjacent upstream or downstream ramps do not affect the prediction of v_{12} for two-lane (one direction) freeway segments, since *all* vehicles are in Lanes 1 and 2. Data have been insufficient to determine whether adjacent ramps influence lane distribution on four-lane (one direction) freeway segments, and thus no such impact is used in this methodology.

Where an upstream or downstream adjacent off-ramp exists on a six-lane freeway, a determination as to whether the ramp is close enough to the subject merge area to influence the area's operation is necessary. The determination is made by finding the equilibrium separation distance L_{EQ}. If the actual distance is larger than or equal to L_{EQ}, Equation 13-3 should be used. If the actual distance is shorter than L_{EQ}, then Equation 13-4 or Equation 13-5 should be used as appropriate.

Exhibit 13-6
Models for Predicting P_{FM} at On-Ramps or Merge Areas

No. of Freeway Lanes[a]	Model(s) for Determining P_{FM}	
4	$P_{FM} = 1.000$	
6	$P_{FM} = 0.5775 + 0.000028\, L_A$	Equation 13-3
	$P_{FM} = 0.7289 - 0.0000135\,(v_F + v_R) - 0.003296 S_{FR} + 0.000063 L_{\mathrm{UP}}$	Equation 13-4
	$P_{FM} = 0.5487 + 0.2628\,(v_D / L_{\mathrm{DOWN}})$	Equation 13-5
8	For $v_F / S_{FR} \le 72$: $P_{FM} = 0.2178 - 0.000125 v_R + 0.01115\,(L_A / S_{FR})$	
	For $v_F / S_{FR} > 72$: $P_{FM} = 0.2178 - 0.000125 v_R$	

SELECTING EQUATIONS FOR P_{FM} FOR SIX-LANE FREEWAYS

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	On	None	Equation 13-3
None	On	On	Equation 13-3
None	On	Off	Equation 13-5 or 13-3
On	On	None	Equation 13-3
Off	On	None	Equation 13-4 or 13-3
On	On	On	Equation 13-3
On	On	Off	Equation 13-5 or 13-3
Off	On	On	Equation 13-4 or 13-3
Off	On	Off	Equation 13-5 or 13-4 or 13-3

Note: [a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
If an adjacent diverge on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 13-3.

The equilibrium distance is obtained by finding the distance at which Equation 13-3 would yield the same value of P_{FM} as Equation 13-4 or Equation 13-5, as appropriate. This results in the following:

For adjacent upstream off-ramps, use Equation 13-6:

$$L_{EQ} = 0.214(v_F + v_R) + 0.444 L_A + 52.32 S_{FR} - 2{,}403$$

Equation 13-6

For adjacent downstream off-ramps, use Equation 13-7:

$$L_{EQ} = \frac{v_D}{0.1096 + 0.000107 L_A}$$

Equation 13-7

where all terms are as previously defined.

When both adjacent upstream and downstream off-ramps are present, the larger resulting value of P_{FM} *is used.*

A special case exists when both an upstream and a downstream adjacent off-ramp are present. In such cases, two different values of P_{FM} could arise: one from consideration of the upstream ramp and the other from consideration of the downstream ramp (they cannot be considered simultaneously). In such cases, the analysis resulting in the larger value of P_{FM} is used.

When an adjacent off-ramp to a merge area on a six-lane freeway is not a one-lane, right-side off-ramp, apply Equation 13-3.

In addition, the algorithms used to include the impact of an upstream or downstream off-ramp on a six-lane freeway are only valid for single-lane, right-side adjacent ramps. Where adjacent off-ramps consist of two-lane junctions or major diverge configurations, or where they are on the left side of the freeway, Equation 13-3 is always applied.

Estimating Flow in Lanes 1 and 2 for Off-Ramps (Diverge Areas)

When approaching an off-ramp (diverge area), all off-ramp traffic must be in freeway Lanes 1 and 2 immediately upstream of the ramp to execute the desired

maneuver. Thus, for off-ramps, the flow in Lanes 1 and 2 consists of all off-ramp vehicles and a proportion of freeway through vehicles, as in Equation 13-8:

Equation 13-8

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

where

v_{12} = flow rate in Lanes 1 and 2 of the freeway immediately upstream of the deceleration lane (pc/h),

v_R = flow rate on the off-ramp (pc/h), and

P_{FD} = proportion of diverging traffic remaining in Lanes 1 and 2 immediately upstream of the deceleration lane.

For off-ramps, the point at which flows are defined is the beginning of the deceleration lane(s), regardless of whether this point is within or outside the ramp influence area.

Exhibit 13-7 contains the equations used to estimate P_{FD} at off-ramp diverge areas. As was the case for on-ramps (merge areas), the value of P_{FD} for four-lane freeways is trivial, since only Lanes 1 and 2 exist.

Exhibit 13-7
Models for Predicting P_{FD} at Off-Ramps or Diverge Areas

No. of Freeway Lanes[a]	Model(s) for Determining P_{FD}
4	$P_{FD} = 1.000$
6	$P_{FD} = 0.760 - 0.000025v_F - 0.000046v_R$ (Equation 13-9) $P_{FD} = 0.717 - 0.000039v_F + 0.604(v_U / L_{UP})$ (Equation 13-10) $P_{FD} = 0.616 - 0.000021v_F + 0.124(v_D / L_{DOWN})$ (Equation 13-11)
8	$P_{FD} = 0.436$

SELECTING EQUATIONS FOR P_{FD} FOR SIX-LANE FREEWAYS

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	Off	None	Equation 13-9
None	Off	On	Equation 13-9
None	Off	Off	Equation 13-11 or 13-9
On	Off	None	Equation 13-10 or 13-9
Off	Off	None	Equation 13-9
On	Off	On	Equation 13-10 or 13-9
On	Off	Off	Equation 13-11, 13-10, or 13-9
Off	Off	On	Equation 13-9
Off	Off	Off	Equation 13-11 or 13-9

Note: [a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
If an adjacent ramp on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 13-9.

For six-lane freeways, three equations are presented. Equation 13-9 is the base case for isolated ramps or for cases in which the impact of adjacent ramps can be ignored. Equation 13-10 addresses cases in which there is an adjacent upstream on-ramp, while Equation 13-11 addresses cases in which there is an adjacent downstream off-ramp. Adjacent upstream off-ramps and downstream on-ramps have not been found to have a statistically significant impact on diverge operations and may be ignored. All variables in Exhibit 13-7 are as previously defined.

Insufficient information is available to establish an impact of adjacent ramps on eight-lane freeways (four lanes in each direction). This methodology does not include such an impact.

Where an adjacent upstream on-ramp or downstream off-ramp on a six-lane freeway exists, a determination as to whether the ramp is close enough to the subject off-ramp to affect its operation is necessary. As was the case for on-ramps, this is done by finding the equilibrium distance L_{EQ}. This distance is determined when Equation 13-9 yields the same value of P_{FD} as Equation 13-10 (for adjacent upstream on-ramps) or Equation 13-11 (adjacent downstream off-ramps). When the actual distance between ramps is greater than or equal to L_{EQ}, Equation 13-9 is used. When the actual distance between ramps is less than L_{EQ}, Equation 13-10 or Equation 13-11 is used as appropriate.

For adjacent upstream on-ramps, use Equation 13-12 to find the equilibrium distance:

$$L_{EQ} = \frac{v_u}{0.071 + 0.000023 v_F - 0.000076 v_R}$$

Equation 13-12

For adjacent downstream off-ramps, use Equation 13-13:

$$L_{EQ} = \frac{v_D}{1.15 - 0.000032 v_F - 0.000369 v_R}$$

Equation 13-13

where all terms are as previously defined.

A special case exists when both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present. In such cases, two solutions for P_{FD} may arise, depending on which adjacent ramp is considered (both ramps cannot be considered simultaneously). In such cases, the larger value of P_{FD} is used.

When both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present, the larger resulting value of P_{FD} is used.

As was the case for merge areas, the algorithms used to include the impact of an upstream or downstream ramp on a six-lane freeway are only valid for single-lane, right-side adjacent ramps. Where adjacent ramps consist of two-lane junctions or major diverge configurations, or where they are on the left side of the freeway, Equation 13-9 is always applied.

When an adjacent ramp to a diverge area on a six-lane freeway is not a one-lane, right-side ramp, apply Equation 13-9.

Checking the Reasonableness of the Lane Distribution Prediction

The algorithms of Exhibit 13-6 and Exhibit 13-7 were developed through regression analysis of a large database. Unfortunately, regression-based models may yield unreasonable or unexpected results when applied outside the strict limits of the calibration database, and they may have inconsistencies at their boundaries.

Therefore, it is necessary to apply some limits to predicted values of flow in Lanes 1 and 2 (v_{12}). The following limitations apply to all such predictions:

Reasonableness checks on the value of v_{12}.

1. The average flow per lane in the outer lanes of the freeway (lanes other than 1 and 2) should not be higher than 2,700 pc/h/ln.
2. The average flow per lane in outer lanes should not be higher than 1.5 times the average flow in Lanes 1 and 2.

These limits guard against cases in which the predicted value of v_{12} implies an unreasonably high flow rate in outer lanes of the freeway. When either of these limits is violated, an adjusted value of v_{12} must be computed and used in the remainder of the methodology.

Application to Six-Lane Freeways

On a six-lane freeway (three lanes in one direction), there is only one outer lane to consider. The flow rate in this outer lane (Lane 3) is given by Equation 13-14:

Equation 13-14

$$v_3 = v_F - v_{12}$$

where

v_3 = flow rate in Lane 3 of the freeway (pc/h/ln),

v_F = flow rate on freeway immediately upstream of the ramp influence area (pc/h), and

v_{12} = flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h).

Then, if v_3 is greater than 2,700 pc/h, use Equation 13-15:

Equation 13-15

$$v_{12a} = v_F - 2{,}700$$

If v_3 is greater than 1.5 × (v_{12}/2), use Equation 13-16:

Equation 13-16

$$v_{12a} = \left(\frac{v_F}{1.75}\right)$$

where v_{12a} equals the adjusted flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h) and all other variables are as previously defined.

In cases where both limitations on outer lane flow rate are violated, the result yielding the highest value of v_{12a} is used. The adjusted value replaces the original value of v_{12} and the analysis continues.

Application to Eight-Lane Freeways

On eight-lane freeways, there are two outer lanes (Lanes 3 and 4). Thus, the limiting values cited previously apply to the average flow rate per lane in these lanes. The average flow in these lanes is computed from Equation 13-17:

Equation 13-17

$$v_{av34} = \frac{v_F - v_{12}}{2}$$

where v_{av34} equals the flow rate in outer lanes (pc/h/ln) and all other variables are as previously defined.

Then, if v_{av34} is greater than 2,700, use Equation 13-18:

Equation 13-18

$$v_{12a} = v_F - 5{,}400$$

If v_{av34} is greater than 1.5 × (v_{12}/2), use Equation 13-19:

Equation 13-19

$$v_{12a} = \left(\frac{v_F}{2.50}\right)$$

where all terms are as previously defined.

In cases where both limitations on outer lane flow rate are violated, the result yielding the highest value of v_{12a} is used. The adjusted value replaces the original value of v_{12} and the analysis continues.

Summary of Step 2

At this point, an appropriate value of v_{12} has been computed and adjusted as necessary.

Step 3: Estimate the Capacity of the Ramp–Freeway Junction and Compare with Demand Flow Rates

There are three major checkpoints for the capacity of a ramp–freeway junction:

Locations for checking the capacity of a ramp–freeway junction.

1. The capacity of the freeway immediately downstream of an on-ramp or immediately upstream of an off-ramp,
2. The capacity of the ramp roadway, and
3. The maximum flow rate entering the ramp influence area.

Freeway capacity immediately downstream of an on-ramp or upstream of an off-ramp is usually the controlling capacity factor.

In most cases, the freeway capacity is the controlling factor. Studies (*1*) have shown that the turbulence in the vicinity of a ramp–freeway junction does not diminish the capacity of the freeway.

Failure of a diverge junction is usually caused by a capacity deficiency at the ramp–street terminal or on the off-ramp roadway.

The capacity of the ramp roadway is rarely a factor at on-ramps, but it can play a major role at off-ramp (diverge) junctions. Failure of a diverge junction is most often caused by a capacity deficiency on the off-ramp roadway or at its ramp–street terminal.

While this methodology establishes a maximum desirable rate of flow entering the ramp influence area, exceeding this value does not cause a failure. Instead, it means that operations may be less desirable than indicated by the methodology. At off-ramps, the total flow rate entering the ramp influence area is merely the estimated value of v_{12}. At on-ramps, however, the on-ramp flow also enters the ramp influence area. Therefore, the total flow entering the ramp influence area at an on-ramp is given by Equation 13-20:

Equation 13-20

$$v_{R12} = v_{12} + v_R$$

where v_{R12} is the total flow rate entering the ramp influence area at an on-ramp (pc/h) and all other variables are as previously defined.

Exhibit 13-8 shows capacity values for ramp–freeway junctions. Exhibit 13-9 shows similar values for high-speed ramps on multilane highways and C-D roadways within freeway interchanges. Exhibit 13-10 shows the capacity of ramp roadways.

Exhibit 13-8
Capacity of Ramp–Freeway Junctions (pc/h)

FFS (mi/h)	Capacity of Upstream/Downstream Freeway Segment[a] No. of Lanes in One Direction: 2	3	4	>4	Max. Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Max. Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
≥70	4,800	7,200	9,600	2,400/ln	4,600	4,400
65	4,700	7,050	9,400	2,350/ln	4,600	4,400
60	4,600	6,900	9,200	2,300/ln	4,600	4,400
55	4,500	6,750	9,000	2,250/ln	4,600	4,400

Notes: [a] Demand in excess of these capacities results in LOS F.
[b] Demand in excess of these values alone does not result in LOS F; operations may be worse than predicted by this methodology.

Exhibit 13-9
Capacity of High-Speed Ramp Junctions on Multilane Highways and C-D Roadways (pc/h)

FFS (mi/h)	Capacity of Upstream/Downstream Highway or C-D Segment[a] No. of Lanes in One Direction: 2	3	>3	Max. Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Max. Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
≥60	4,400	6,600	2,200/ln	4,600	4,400
55	4,200	6,300	2,100/ln	4,600	4,400
50	4,000	6,000	2,000/ln	4,600	4,400
45	3,800	5,700	1,900/ln	4,600	4,400

Notes: [a] Demand in excess of these capacities results in LOS F.
[b] Demand in excess of these values alone does not result in LOS F; operations may be worse than predicted by this methodology.

Exhibit 13-10
Capacity of Ramp Roadways (pc/h)

Ramp FFS S_{FR} (mi/h)	Capacity of Ramp Roadway: Single-Lane Ramps	Two-Lane Ramps
>50	2,200	4,400
>40–50	2,100	4,200
>30–40	2,000	4,000
≥20–30	1,900	3,800
<20	1,800	3,600

Note: Capacity of a ramp roadway does not ensure an equal capacity at its freeway or other high-speed junction. Junction capacity must be checked against criteria in Exhibit 13-8 and Exhibit 13-9.

Ramp–Freeway Junction Capacity Checkpoint

As noted previously, it is generally the capacity of the upstream or downstream freeway segment that limits flow through a merge or diverge area, assuming that the number of freeway lanes entering and leaving the ramp junction is the same. In such cases, the critical checkpoint for freeway capacity is

- Immediately downstream of an on-ramp influence area (v_{FO}), or
- Immediately upstream of an off-ramp influence area (v_F).

These are logical checkpoints, since each represents the point at which maximum freeway flow exists.

When a ramp junction or major merge/diverge area involves lane additions or lane drops at the junction, freeway capacity must be checked both immediately upstream and downstream of the ramp influence area.

Failure of any ramp–freeway junction capacity check results in LOS F.

Failure of any ramp–freeway junction capacity check (i.e., demand exceeds capacity: v/c is greater than 1.00) results in LOS F.

Ramp Roadway Capacity Checkpoint

The capacity of the ramp roadway should always be checked against the demand flow rate on the ramp. For on-ramp or merge junctions, this is rarely a problem. Theoretically, cases could exist in which demand exceeds capacity. A

failure due to insufficient on-ramp capacity does not, in itself, create problems on the freeway. Rather, it would result in queuing at the streetside terminal of the ramp (or in the case of a freeway-to-freeway ramp, on the entering freeway).

At off-ramps or diverge areas, the most frequent cause of failure is insufficient capacity on the off-ramp—due to either the ramp roadway or a failure of the ramp–street terminal. This methodology checks only for the off-ramp roadway capacity. The capacity of the ramp–street junction must be evaluated by using appropriate methodologies for unsignalized intersections (Chapter 19, 20, or 21) or signalized interchange ramp terminals (Chapter 22).

If the off-ramp demand flow rate v_R exceeds the capacity of the off-ramp, LOS F prevails. If appropriate analysis results in a finding that the ramp–street terminal is operating at a v/c ratio greater than 1.00 on the ramp approach leg, a queuing analysis should be conducted to evaluate (*a*) the extent of the queue that is likely to exist on the ramp roadway and (*b*) whether the queue is close enough to the ramp–freeway junction to affect its operation negatively.

Maximum Desirable Flow Entering the Ramp Influence Area

While a checkpoint for v_{12} (off-ramps) or v_{R12} (on-ramps) is conducted, failure does not result in assignment of LOS F, unless another failure occurs on a ramp roadway or freeway segment. Failing this checkpoint generally means that there will be more turbulence in the ramp junction influence area than predicted by this methodology. Thus, predicted densities are most likely lower than those that will exist, and predicted speeds are most likely higher than those that will actually occur.

Failure of the check for flow entering the ramp influence area (v_{12}, v_{R12}) does not automatically result in LOS F but does indicate the need for additional interpretation of the results.

Step 4: Estimate Density in the Ramp Influence Area and Determine the Prevailing LOS

Once the flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area is determined, the expected density in the ramp influence area can be estimated.

Density in On-Ramp (Merge) Influence Areas

The density in on-ramp influence areas is estimated with Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

Equation 13-21

where D_R is the density in the ramp influence area (pc/mi/ln) and all other variables are as previously defined.

The equation is logical. As more on-ramp vehicles and freeway vehicles in Lanes 1 and 2 enter the ramp influence area, its density is expected to increase. As the length of the acceleration lane increases, there is more physical space in the ramp influence area, and operating speeds of merging vehicles are expected to increase—both tending to reduce densities.

Density in Off-Ramp (Diverge) Influence Areas

The density in off-ramp influence areas is estimated with Equation 13-22:

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

Equation 13-22

where all variables are as previously defined.

This equation also follows logical trends. There is no separate term for v_R because it is included in v_{12} for off-ramps. As the number of vehicles entering the ramp influence area increases, density increases. As the length of the deceleration lane increases, the additional space provided and the resulting higher speeds of merging vehicles both act to reduce density.

Determining LOS

LOS in ramp influence areas is directly related to the estimated density within the area, as given by Equation 13-21 or Equation 13-22. Exhibit 13-2, shown previously, contains the criteria for this determination. Note again that density definitions of LOS apply only to stable flow (i.e., LOS A–E). LOS F exists only when the capacity of the ramp junction is insufficient to accommodate the existing or projected demand flow rate.

If it is determined that a merge or diverge segment is operating (or expected to operate at) LOS F, the analyst should go to Chapter 10, Freeway Facilities, and conduct a facility analysis that will estimate the spatial and time impacts of queuing resulting from the breakdown.

Step 5: Estimate Speeds in the Vicinity of Ramp–Freeway Junctions

While an estimation of average vehicle speeds in and adjacent to ramp influence areas is not necessary, it is often a useful additional performance measure. Two types of speeds may be estimated:

- Average speed of vehicles within the ramp influence area (mi/h), and
- Average speed of vehicles across all lanes (including outer lanes) within the 1,500-ft length of the ramp influence area (mi/h).

Both types of speeds are needed when a freeway facility analysis is conducted (Chapter 10), while the first type of speed provides a useful companion measure to density within the ramp influence area in all cases.

Exhibit 13-11 and Exhibit 13-12 provide equations for estimating the average speed of vehicles (*a*) within the ramp influence area and (*b*) in outer lanes of the freeway adjacent to the 1,500-ft ramp influence area. For four-lane freeways (two lanes in each direction), there are no "outer lanes." For six-lane freeways (three lanes in each direction), there is one outer lane (Lane 3). For eight-lane freeways (four lanes in each direction), there are two outer lanes (Lanes 3 and 4). Exhibit 13-13 provides equations to determine the average speed of all vehicles (ramp plus all freeway vehicles) within the 1,500-ft length of the ramp influence area.

Exhibit 13-11
Estimating Speed at On-Ramp (Merge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS - (FFS - 42)M_S$	
	$M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002\ (L_A S_{FR}/1{,}000)$	
Outer lanes of freeway	$S_O = FFS$	$v_{OA} < 500$ pc/h
	$S_O = FFS - 0.0036(v_{OA} - 500)$	500 pc/h $\le v_{OA} \le$ 2,300 pc/h
	$S_O = FFS - 6.53 - 0.006(v_{OA} - 2{,}300)$	$v_{OA} > 2{,}300$ pc/h

Exhibit 13-12
Estimating Speed at Off-Ramp (Diverge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS - (FFS - 42)D_S$ $D_S = 0.883 + 0.00009v_R - 0.013S_{FR}$	
Outer lanes of freeway	$S_O = 1.097FFS$	$v_{OA} < 1{,}000$ pc/h
	$S_O = 1.097FFS - 0.0039\left(v_{OA} - 1{,}000\right)$	$v_{OA} \geq 1{,}000$ pc/h

Exhibit 13-13
Estimating Average Speed of All Vehicles at Ramp–Freeway Junctions

Value	Equation
Average flow in outer lanes v_{OA} (pc/h)	$v_{OA} = \dfrac{v_F - v_{12}}{N_O}$
Average speed for on-ramp (merge) junctions (mi/h)	$S = \dfrac{v_{R12} + v_{OA}N_O}{\left(\dfrac{v_{R12}}{S_R}\right) + \left(\dfrac{V_{OA}N_O}{S_O}\right)}$
Average speed for off-ramp (diverge) junctions (mi/h)	$S = \dfrac{v_{12} + v_{OA}N_O}{\left(\dfrac{v_{12}}{S_R}\right) + \left(\dfrac{v_{OA}N_O}{S_O}\right)}$

While many (but not all) of the variables in Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 have been defined previously, all are defined here for convenience:

S_R = average speed of vehicles within the ramp influence area (mi/h); for merge areas, this includes all ramp and freeway vehicles in Lanes 1 and 2; for diverge areas, this includes all vehicles in Lanes 1 and 2;

S_O = average speed of vehicles in outer lanes of the freeway, adjacent to the 1,500-ft ramp influence area (mi/h);

S = average speed of all vehicles in all lanes within the 1,500-ft length covered by the ramp influence area (mi/h);

FFS = free-flow speed of the freeway (mi/h);

S_{FR} = free-flow speed of the ramp (mi/h);

L_A = length of acceleration lane (ft);

L_D = length of deceleration lane (ft);

v_R = demand flow rate on ramp (pc/h);

v_{12} = demand flow rate in Lanes 1 and 2 of the freeway immediately upstream of the ramp influence area (pc/h);

v_{R12} = total demand flow rate entering the on-ramp influence area, including v_{12} and v_R (pc/h);

v_{OA} = average demand flow per lane in outer lanes adjacent to the ramp influence area (not including flow in Lanes 1 and 2) (pc/h/ln);

v_F = demand flow rate on freeway immediately upstream of the ramp influence area (pc/h);

N_O = number of outer lanes on the freeway (1 for a six-lane freeway; 2 for an eight-lane freeway);

M_S = speed index for on-ramps (merge areas); this is simply an intermediate

computation that simplifies the equations; and

D_S = speed index for off-ramps (diverge areas); this is simply an intermediate computation that simplifies the equations.

Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 only apply to stable flow conditions. Consult Chapter 10 for analysis of oversaturated conditions.

The equations in Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 apply only to cases in which operation is stable (LOS A–E). Analysis of operational details for cases in which LOS F is present relies on deterministic queuing approaches, as presented in Chapter 10, Freeway Facilities.

Flow rates in outer lanes can be higher than the value cited for basic freeway segments. The basic freeway segment values represent averages across all freeway lanes, not flow rates in a single lane or a subset of lanes. The methodology herein allows flows in outer lanes to be as high as 2,700 pc/h/ln. The equations for average speed in outer lanes were based on a database that included average outer lane flows as high as 2,988 pc/h/ln while still maintaining stable flow. Values over 2,700 pc/h/ln, however, are unusual and cannot be expected in the majority of situations.

In addition, the equations of Exhibit 13-11 do not allow a predicted speed over the FFS for merge areas. For diverge areas at low flow rates, however, the average speed in outer lanes may marginally exceed the FFS. As with average lane flow rates, the FFS is stated as an average across all lanes, and speeds in individual lanes can exceed this value. Despite this, the average speed of all vehicles *S* should be limited to a maximum value equal to the FFS.

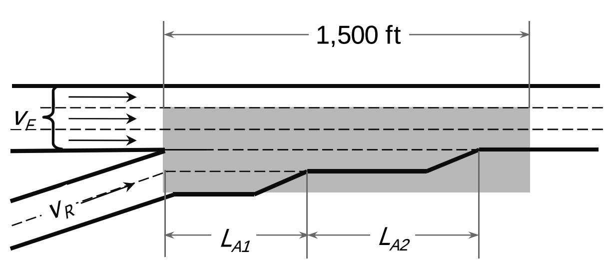
SPECIAL CASES

As noted previously, the computational procedure for ramp–freeway junctions was calibrated for single-lane, right-side ramps. Many other merge and diverge configurations may be encountered, however. In these cases, the general methodology is modified to account for special situations. These modifications are discussed in the sections that follow.

Two-Lane On-Ramps

Exhibit 13-14 illustrates the geometry of a typical two-lane ramp–freeway junction. It is characterized by two separate acceleration lanes, each successively forcing merging maneuvers to the left.

Exhibit 13-14
Typical Geometry of a Two-Lane Ramp–Freeway Junction



Two-lane on-ramps entail two modifications to the basic methodology: the flow remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area is generally somewhat higher than it is for one-lane on-ramps in similar situations, and densities in the merge influence area are lower than those for similar one-lane on-ramp situations. The lower density is primarily due to the

existence of two acceleration lanes and the generally longer distance over which these lanes extend. Thus, two-lane on-ramps handle higher ramp flows more smoothly and at a better LOS than if the same flows were carried on a one-lane ramp–freeway junction.

Two-lane on-ramp–freeway junctions, however, do not enhance the capacity of the junction. The downstream freeway capacity still controls the total output capacity of the merge area, and the maximum desirable number of vehicles entering the ramp influence area is not changed.

There are three computational modifications to the general methodology for two-lane on-ramps.

First, while v_{12} is still estimated as $v_F \times P_{FM}$, the values of P_{FM} are modified as follows:

- For four-lane freeways: P_{FM} = 1.000;
- For six-lane freeways: P_{FM} = 0.555; and
- For eight-lane freeways: P_{FM} = 0.209.

Second, in all equations using the length of the acceleration lane L_A, this value is replaced by the effective length of both acceleration lanes L_{Aeff} from Equation 13-23:

$$L_{Aeff} = 2L_{A1} + L_{A2}$$

Equation 13-23

A two-lane ramp is always considered to be isolated (i.e., no adjacent ramp conditions affect the computation).

Component lengths are as illustrated in Exhibit 13-14.

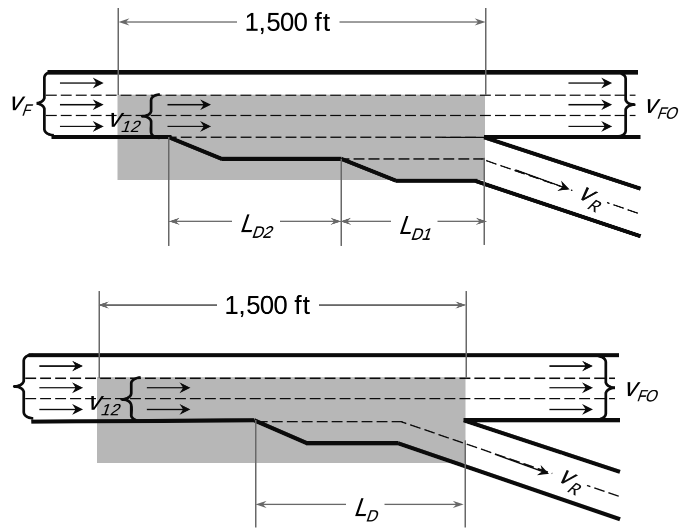
Two-Lane Off-Ramps

Two common types of diverge geometries are in use with two-lane off-ramps, as shown in Exhibit 13-15. In the first, two successive deceleration lanes are introduced. In the second, a single deceleration lane is used. The left-hand ramp lane splits from Lane 1 of the freeway at the gore area, without a deceleration lane.

As is the case for two-lane on-ramps, there are three computational step modifications. While v_{12} is still computed as $v_R + (v_F - v_R) \times P_{FD}$, the values of P_{FD} are modified as follows:

- For four-lane freeways: P_{FD} = 1.000;
- For six-lane freeways: P_{FD} = 0.450; and
- For eight-lane freeways: P_{FD} = 0.260.

Exhibit 13-15
Common Geometries for Two-Lane Off-Ramp–Freeway Junctions



Where a single deceleration lane is used, there is no modification to the length of the deceleration lane L_D; where two deceleration lanes exist, the length is replaced by the effective length L_{Deff} in all equations, obtained from Equation 13-24:

Equation 13-24

$$L_{Deff} = 2L_{D1} + L_{D2}$$

A two-lane ramp is always considered to be isolated (i.e., no adjacent ramp conditions affect the computation).

Component lengths are as illustrated in Exhibit 13-15.

The capacity of a two-lane off-ramp is essentially equal to that of a similar one-lane off-ramp.

The capacity of a two-lane off-ramp freeway junction is essentially equal to that of a similar one-lane off-ramp; that is, the total flow capacity through the diverge is unchanged. It is limited by the upstream freeway, the downstream freeway, or the off-ramp capacity. While the capacity is not affected by the presence of two-lane junctions, the lane distribution of vehicles is more flexible than in a similar one-lane case. The two-lane junction may also be able to accommodate a higher off-ramp flow than can a single-lane off-ramp.

Left-Hand On- and Off-Ramps

While they are not normally recommended, left-hand ramp–freeway junctions do exist on some freeways, and they occur frequently on C-D roadways. The left-hand ramp influence area covers the same 1,500-ft length as that of right-hand ramps—upstream of off-ramps; downstream of on-ramps.

For right-hand ramps, the ramp influence area involves Lanes 1 and 2 of the freeway. For left-hand ramps, the ramp influence area involves the two leftmost lanes of the freeway. For four-lane freeways (two lanes in each direction), this does not involve any changes, since only Lanes 1 and 2 exist. For six-lane freeways (three lanes in each direction), the flow in Lanes 2 and 3 (v_{23}) is involved. For eight-lane freeways (four lanes in each direction), the flow in Lanes 3 and 4 (v_{34}) is involved.

While there is no direct methodology for the analysis of left-hand ramps, some rational modifications can be applied to the right-hand ramp methodology to produce reasonable results (*3*).

It is suggested that analysts compute v_{12} as if the ramp were on the right. An estimate of the appropriate flow rate in the two leftmost lanes is then obtained by multiplying the result by the adjustment factors shown in Exhibit 13-16.

Exhibit 13-16
Adjustment Factors for Left-Hand Ramp–Freeway Junctions

Freeway Size	Adjustment Factor for Left-Hand Ramps On-Ramps	Off-Ramps
Four-lane	1.00	1.00
Six-lane	1.12	1.05
Eight-lane	1.20	1.10

The remaining computations for density and speed continue by using the value of v_{23} (six-lane freeways) or v_{34} (eight-lane freeways), as appropriate. All capacity values remain unchanged.

Ramp–Freeway Junctions on 10-Lane Freeways (Five Lanes in Each Direction)

Freeway segments with five continuous lanes in a single direction are becoming more common in North America. A procedure is therefore needed to analyze a single-lane, right-hand on- or off-ramp on such a segment.

The approach taken is relatively simple: estimate the flow in Lane 5 of such a segment and deduct it from the approaching freeway flow v_F. With the Lane 5 flow deducted, the segment can now be treated as if it were an eight-lane freeway (*4*). Exhibit 13-17 shows the recommended values for flow rate in Lane 5 of these segments.

Exhibit 13-17
Expected Flow in Lane 5 of a 10-Lane Freeway Immediately Upstream of a Ramp–Freeway Junction

On-Ramps: Approaching Freeway Flow v_F (pc/h)	On-Ramps: Approaching Lane 5 Flow v_5 (pc/h)	Off-Ramps: Approaching Freeway Flow v_F (pc/h)	Off-Ramps: Approaching Lane 5 Flow v_5 (pc/h)
≥8,500	2,500	≥7,000	0.200 v_F
7,500–8,499	0.285 v_F	5,500–6,999	0.150 v_F
6,500–7,499	0.270 v_F	4,000–5,499	0.100 v_F
5,500–6,499	0.240 v_F	<4,000	0
<5,500	0.220 v_F		

Once the expected flow in Lane 5 is determined, the effective total freeway flow rate in the remaining four lanes is computed from Equation 13-25:

Equation 13-25

$$v_{F4eff} = v_F - v_5$$

where

v_{F4eff} = effective approaching freeway flow in four lanes (pc/h),

v_F = total approaching freeway flow in five lanes (pc/h), and

v_5 = estimated approaching freeway flow in Lane 5 (pc/h).

The remainder of the analysis uses the adjusted approaching freeway flow rate and treats the geometry as if it were a single-lane, right-hand ramp junction on an eight-lane freeway (four lanes in each direction).

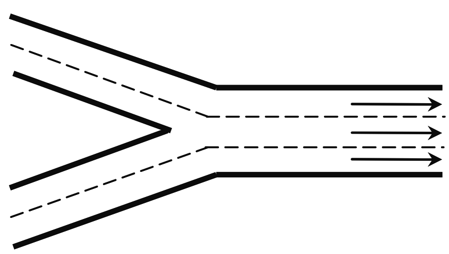
There is no calibrated procedure for adapting the methodology of this chapter to freeways with more than five lanes in one direction. The approach of Equation 13-25 is, however, conceptually adaptable to such situations. A local calibration of the amount of traffic using Lanes 5+ would be needed. The remaining flow could then be modeled as if it were taking place on a four-lane (one direction) segment.

Major Merge Areas

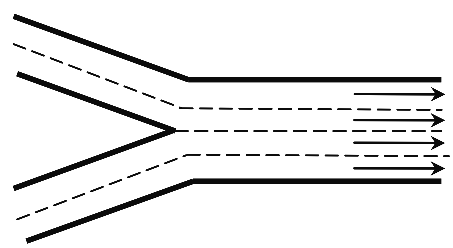
A major merge area is one in which two primary roadways, each having multiple lanes, merge to form a single freeway segment. Such junctions occur when two freeways join to form a single freeway or when a major multilane high-speed ramp joins with a freeway. Major merges are different from one- and two-lane on-ramps in that each of the merging roadways is generally at or near freeway design standards and no clear ramp or acceleration lane is involved in the merge.

Such merge areas come in a variety of geometries, all of which fall into one of two categories. In one geometry, the number of lanes leaving the merge area is one less than the total number of lanes entering it. In the other, the number of lanes leaving the merge area is the same as that entering it. These geometries are illustrated in Exhibit 13-18.

Exhibit 13-18
Major Merge Areas Illustrated



(a) Major Merge with One Lane Dropped



(b) Major Merge with No Lane Dropped

LOS cannot be determined for major merge areas.

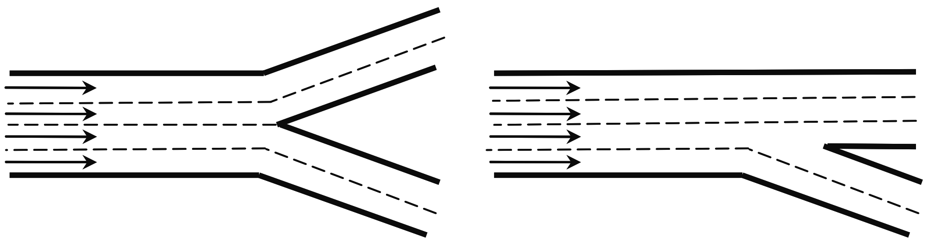
There are no effective models of performance for a major merge area. Therefore, analysis is limited to checking capacities on the approaching legs and the downstream freeway segment. A merge failure would be indicated by a v/c ratio in excess of 1.00. LOS cannot be determined for major merge areas. Problems in major merge areas usually result from insufficient capacity of the downstream freeway segment.

Major Diverge Areas

The two common geometries for major diverge areas are illustrated in Exhibit 13-19. In the first case, the number of lanes leaving the diverge area is the same as the number entering it. In the second, the number of lanes leaving the diverge area is one more than the number entering it.

The principal analysis of a major diverge area involves checking the capacity of entering and departing roadways, all of which are generally built to mainline standards. A failure results when any of the demand flow rates exceeds the capacity of the segment.

Exhibit 13-19
Major Diverge Areas Illustrated



(a) Major Diverge Area with No Lane Addition (b) Major Diverge Area with Lane Addition

For major diverge areas, a model exists for computing the average density across all approaching freeway lanes within 1,500 ft of the diverge, as given in Equation 13-26:

Equation 13-26

$$D_{MD} = 0.0175\left(\frac{v_F}{N}\right)$$

where

D_{MD} = density in the major diverge influence area (which includes all approaching freeway lanes) (pc/mi/ln),

v_F = demand flow rate immediately upstream of the major diverge influence area (pc/h), and

N = number of lanes approaching the major diverge.

The result can be compared with the criteria of Exhibit 13-2 to determine a LOS for the major diverge influence area. Note that the density and LOS estimates are only valid for stable cases (i.e., not in cases in which LOS F exists because of a capacity deficiency on the approaching or departing legs of the diverge).

Effect of Ramp Control at On-Ramps

For the purposes of this methodology, procedures are not modified in any way to account for the local effect of ramp control—except for the limitation that the ramp meter may have on the ramp demand flow rate. Research (*5*) has found that the breakdown of a merge area may be a probabilistic event based on the platoon characteristics of the arriving ramp vehicles. Ramp meters facilitate uniform gaps between entering ramp vehicles and may reduce the probability of a breakdown on the associated freeway mainline.

OVERLAPPING RAMP INFLUENCE AREAS

Whenever a series of ramps on a freeway is analyzed, the 1,500-ft ramp influence areas could overlap. In such cases, the operation in the overlapping region is determined by the ramp influence area having the highest density.

3. APPLICATIONS

The methodology of this chapter is most often used to estimate the capacity and LOS of ramp–freeway junctions. The steps are most easily applied in the operational analysis mode (i.e., all traffic and roadway conditions are specified), and the capacity (and v/c ratio) and expected LOS are found. Other types of analysis, however, are possible.

DEFAULT VALUES

A comprehensive presentation of potential default values for uninterrupted-flow facilities is provided elsewhere (*6*). Chapter 10, Freeway Facilities, provides a summary of the default values for freeways. These defaults cover the key characteristics of peak hour factor (PHF) and percent heavy vehicles (%HV) on freeways. Recommendations are based on geographical region, population, and time of day. All general freeway default values may be applied to the analysis of ramp–freeway junctions in the absence of field data or projections of conditions.

Because of the number of variables involved in the analysis of ramps, which have been discussed previously, it is difficult to base an analysis on too many default values. Clearly, all demand flow rates must be specified, even if they are projections.

Ramp geometric characteristics cover a variety of conditions; default values should be avoided if possible.

Similarly, geometric characteristics of ramps cover a wide variety of conditions. If absolutely necessary, the following additional default values may be applied to a ramp junction analysis:

- Length of acceleration lane L_A = 800 ft,
- Length of deceleration lane L_D = 400 ft,
- FFS of ramp S_{FR} = 35 mi/h, and
- Driver population factor f_P = 1.00.

Obviously, as the number of default values used in any analysis increases, the accuracy of the result becomes more approximate, and the result may be significantly different from the actual outcome (depending on local conditions). If locally calibrated default values are available, they may be substituted for the values above.

ESTABLISH ANALYSIS BOUNDARIES

No ramp–freeway junction is completely isolated. However, for the purposes of this methodology, many may operate as if they were. In the analysis of ramp–freeway junctions, it is important to establish the segment of freeway over which ramp junctions are to be analyzed. Once this is done, each ramp may be analyzed in conjunction with the possible impacts of upstream and downstream adjacent ramps according to the methodology.

Analysis boundaries may also include different demand scenarios related to the time of the day or to different development scenarios that produce different demand flow rates.

Any application of the methodology presented in this chapter can be made easier by carefully defining the spatial and time boundaries of the analysis.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design analysis.

Operational Analysis

Operational analysis determines density, LOS, and speed within the ramp influence area for a specified set of conditions.

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including

- Analysis hour demand volumes for the subject ramp, adjacent ramps, and freeway (veh/h);
- Heavy vehicle percentages for all component demand volumes (ramps, adjacent ramps, freeway);
- PHF for all component demand volumes (ramp, adjacent ramps, freeway);
- Freeway terrain (level, rolling, mountainous, specific grade);
- FFS of the freeway and ramp (mi/h);
- Ramp geometrics: number of lanes, terrain, length of acceleration lane(s) or deceleration lane(s); and
- Distance to upstream and downstream adjacent ramps (ft).

The outputs of an operational analysis will be estimates of density, LOS, and speed for the ramp influence area. The capacity of the ramp–freeway junction will also be established.

The steps of the methodology, described in the Methodology section, are to be followed directly without modification.

Design Analysis

Design analysis seeks to determine the geometric characteristics of the ramp that are needed to deliver a target LOS.

In design analysis, a target LOS is set and all relevant demand volumes are specified. The analysis seeks to determine the geometric characteristics of the ramp that are needed to deliver the target LOS. These characteristics include

- FFS of the ramp S_{FR} (mi/h),
- Length of acceleration L_A or deceleration lane L_D (ft), and
- Number of lanes on the ramp.

In some cases, variables such as the type of junction (e.g., major merge, two-lane) may also be under consideration.

There is no convenient way to compute directly the optimal value of any one variable without specifying all of the others. Even then, the computational methodology does not easily create the desired result.

Therefore, most design analysis becomes a trial-and-error application of the operational analysis procedure. Individual characteristics can be incrementally

changed, as can groups of characteristics, to find scenarios that produce the desired LOS.

In many cases, some of the variables may be fixed by site-specific conditions. These can be set at their limiting values before attempting to optimize the others.

It is possible to program a spreadsheet to complete such an analysis, providing scenario results by simply changing some of the input variables under consideration. HCM-implementing software can also be used to simplify the computational process.

Planning and Preliminary Engineering Analysis

Planning and preliminary engineering analysis also seeks to determine the geometric characteristics of the ramp that are needed to deliver a target LOS, but it relies on more general input data.

The desired outputs of planning and preliminary engineering analysis are virtually the same as those for design analysis. The primary difference is that planning and preliminary engineering analysis occurs very early in the process of project consideration.

The first criterion that categorizes such applications is the need to use more general estimates of input data. Many of the default values specified for freeway facilities in Chapter 10 would be applied; alternatively, local default values can be substituted. Demand volumes might be specified only as expected values of annual average daily traffic (AADT) for a target year. Directional design-hour volumes are based on AADTs; default (local or global) values are used for the *K*-factor (the proportion of AADT occurring in the peak hour) and the *D*-factor (the proportion of peak hour traffic traveling in the peak direction). Guidance on these values is given in Chapter 3, Modal Characteristics.

On the basis of these default and estimated values, the analysis is conducted in the same manner as a design analysis.

Service Volumes and Service Flow Rates

The method can be applied to determine service volumes for LOS A–E for a specified set of conditions.

Service volume is the maximum hourly volume that can be accommodated without exceeding the limits of the various levels of service during the worst 15 min of the analysis hour. Service volumes can be found for LOS A–E. LOS F, which represents unstable flow, does not have a service volume.

Service flow rates are the maximum rates of flow (within a 15-min period) that can be accommodated without exceeding the limits of the various levels of service. As is the case for service volumes, service flow rates can be found for LOS A–E, but none is defined for LOS F. The relationship between a service volume and a service flow rate is as follows:

Equation 13-27

$$SV_i = SF_i \times PHF$$

where

SV_i = service volume for LOS i (pc/h),

SF_i = service flow rate for LOS i (pc/h), and

PHF = peak hour factor.

For ramp–freeway junctions, service flow rate or service volume could be defined in several ways. It might be argued that since ramp–freeway junction capacities are usually limited by the upstream or downstream freeway segment,

service flow rates and service volumes should be based on basic freeway criteria applied to the upstream or downstream freeway segments. This, however, would ignore the levels of service defined for the ramp influence area, which are the only unique service descriptors for ramps.

Levels of service for ramp–freeway junctions are defined in Exhibit 13-2 and relate to the density within the ramp influence area. The methodology estimates this density by using a series of algorithms affected by demand flows on the freeway, ramp, and adjacent ramps; ramp geometrics; and distances to adjacent ramps. The methodology uses demand volumes in vehicles per hour converted to demand flow rates in passenger cars per hour. Therefore, service flow rates and service volumes would originally be estimated in terms of flow rates in passenger cars per hour. They would then be converted back to demand volumes in vehicles per hour.

Because the balance of ramp and freeway demands has a significant impact on densities, there are a number of ways in which service flow rates and volumes can be considered:

- The limiting total upstream demand volume that produces a given LOS within the ramp influence area. The split between arriving freeway volume and ramp volume would have to be specified.
- The limiting volume entering the ramp influence area that produces a given LOS within the ramp influence area. Since this relies on the approaching freeway volume, the split between freeway and ramp demand would still have to be specified.
- The limiting ramp volume that produces a given LOS within the ramp influence area, based on a fixed upstream freeway demand.

Any of these are viable concepts for establishing a ramp service flow rate or service volume.

A number of factors influence the service volume or flow rate result; each situation must be individually considered.

In addition to different ways of interpreting a service volume or service flow rate, a large number of characteristics will influence the result, including the PHF, %HV, length of acceleration or deceleration lane(s), ramp FFS, and any relevant data for adjacent ramps. It is, therefore, virtually impossible to define a representative "typical" case with broadly applicable results. Each case must be individually considered.

The Example Problems section includes an example of how ramp junction service flow rates and service volumes can be computed.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for applying alternative tools to the analysis of ramps and ramp junctions. Additional information on this topic may be found in the Volume 4 Technical Reference Library.

The HCM methodology for analyzing merge and diverge segments estimates the density of the ramp influence area (which includes the two rightmost lanes of the freeway and the acceleration or deceleration lane) and provides the

respective LOS. As an intermediate step, the methodology estimates the capacity at various points through the section, and if the capacity is exceeded, the LOS is determined to be F without further calculation of density. The methodology is primarily based on the estimation of the demand into the influence area v_{12}.

Strengths of the HCM Procedure

This chapter's procedures were developed on the basis of extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent a body of expert consensus. Most simulation packages will not include the level of detail present in this methodology concerning the ramp itself and its adjacent upstream and downstream ramps.

The HCM procedure's strengths are as follows:

- The methodology provides capacity estimates. Simulators do not provide capacity estimates directly; they can be obtained by devising a data collection scheme in the simulator. Furthermore, the user can modify those simulated capacities by modifying specific input values, such as the minimum acceptable headway.
- The methodology explicitly considers the impacts of the presence of and demands on the upstream and downstream ramps.
- It produces a single deterministic estimate of density, which is important for some purposes, such as development impact review.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

A list of the HCM's limitations for freeway merge and diverge segments is provided in Exhibit 13-20.

Exhibit 13-20
Limitations of the HCM Ramps and Ramp Junctions Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Managed lanes, such as HOV lanes, as ramp entrance lanes	Modeled explicitly by simulation
Ramp metering	Modeled explicitly by simulation
Oversaturated conditions (*Refer to Chapter 10 for further discussion*)	Modeled explicitly by simulation
Posted speed limit and extent of police enforcement	Can be approximated by using assumptions related to the desired speed along a given segment
Presence of intelligent transportation system features	Several features modeled explicitly by simulation; others may be approximated by using assumptions (for example, by modifying origin–destination demands by time interval)
Freeway operational analysis beyond the 1,500-ft area of influence	Modeled explicitly by simulation
Capacity-enhancing effects of ramp metering	Can be approximated by using assumptions related to car-following, lane-changing, and gap-acceptance behavior

Ramp junctions can also be analyzed with a variety of stochastic and deterministic simulation packages that address freeways. These packages can be useful in analyzing the extent of congestion when there are failures either within or downstream of the simulated facility range.

Additional Features and Performance Measures Available From Alternative Tools

This chapter provides a methodology for estimating the capacity, speed, and density in the area of influence of on- and off-ramps, given traffic demands and segment characteristics. Alternative tools offer additional performance measures including delay, stops, queue lengths, fuel consumption, pollution, and operating costs.

As with most other HCM procedural chapters, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of downstream conditions on lane utilization and backup beyond the segment boundary is a good example of a situation that can benefit from the increased insight offered by a microscopic model.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The subject of performance-measure comparisons was discussed in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results. This section deals with topics that apply specifically to ramps and ramp junctions.

Most simulation packages do not provide separate density estimates for the two right-hand lanes within a link, which is a potentially significant obstacle in obtaining service measures.

When alternative tools are used, the analyst must be careful to note the definitions of simulation outputs. This chapter's measure of effectiveness for ramps and ramp junctions is the density of the ramp influence area. However, most simulators do not provide density estimates separately for the two rightmost lanes within a link. This is a potentially significant obstacle in obtaining the service measures for ramp junctions from a simulator (unless the freeway has only two lanes per direction). Furthermore, in a simulator, there are lane changes along the entire segment. Therefore, it is not clear how a simulator should address the partial presence of vehicles in the link to ensure compatibility with the HCM. Also, as is generally the case for basic freeway segments, increased speed variability in driver behavior (which simulators usually include) results in lower average space mean speed and higher density.

In obtaining density from alternative models, it is important to consider the following:

- The ability of the simulator to provide density for the two rightmost lanes of the freeway;
- The vehicles included in the density estimation and how partial presence of vehicles on the link is considered;
- The manner in which the acceleration and deceleration lanes are considered in the density estimation;
- The units used by the simulator to measure density [most use vehicles rather than passenger cars; converting vehicles to passenger cars by using

the HCM's passenger-car equivalence (PCE) values is typically not appropriate, given that simulator assumptions with regard to heavy vehicle performance vary widely];

- The units used in the reporting of density (i.e., whether density is reported per lane mile);
- The homogeneity of the analysis segment in the simulator, as the HCM assumes conditions to be homogeneous (unless it is a specific upgrade or downgrade segment, in which case the segment length is used to estimate the PCE values); and
- The treatment of driver variability by the simulator, as increased driver variability in the simulator will generally increase the average density.

With regard to capacity, the HCM provides capacity estimates in units of passenger cars per hour per lane as a function of FFS for the locations approaching and departing the merge junction. In comparing the HCM estimates with capacity estimates from a simulator, the following should be considered:

- The manner in which a simulator provides the number of vehicles exiting a segment. In some cases it may be necessary to provide virtual detectors at specific points on the simulated segment so that the maximum throughput can be obtained.
- The simulator provides the maximum throughput at a particular location in units of vehicles, rather than passenger cars. Converting these units to passenger cars by using the HCM's PCE values is typically not appropriate, given that simulator assumptions with regard to heavy vehicle performance vary widely.
- A simulator will likely include inputs such as the "minimum separation of vehicles," which greatly affects the maximum throughput.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Ramp junction density does not change with FFS in the HCM method, but density is a function of FFS in most simulation packages.

In the HCM, the density at a ramp junction does not change with FFS, although density drops as a function of FFS on basic freeway segments. In simulators, the density typically changes as a function of FFS (or the desired speed). Therefore, calibration of a site using a specific FFS does not necessarily ensure that the site will be calibrated for a different FFS. Capacity, on the other hand, increases in the HCM with increasing FFS, which is typically the case with simulators.

The HCM method is based on the estimated demand approaching the ramp influence area. This demand is estimated as a function of the presence of and demands on the upstream and downstream ramps. Traffic simulators do not typically allow the user to input the specific percentages of traffic on each lane at the beginning of a link. Their internal rules relative to the lane chosen by a vehicle in a given link vary widely and can be modified by changing various default values within the simulator. In some simulators, virtual vehicles are "aware" of their ultimate destination; in others, the exit choice is made on a link-by-link basis. Therefore, in comparing HCM results with those of a simulator, the

analyst should, as an intermediate check, compare the flow approaching the two rightmost lanes of the junction.

Adjustment of Simulation Parameters to the HCM Results

The most important elements to be adjusted in analyzing a ramp junction are as follows:

- The flow approaching the two rightmost lanes (this is an intermediate step but would ensure that the influence of upstream and downstream ramps is considered in a manner compatible with the HCM), and
- The capacity of the junction at the critical locations indicated in the HCM (i.e., downstream of the junction and approaching the influence area).

Step-by-Step Recommendations for Applying Alternative Tools

The following steps are recommended when an alternative tool is applied to the analysis of ramps and ramp junctions:

1. Determine whether the chosen tool can provide density for the two rightmost lanes of the freeway and what approach is used to obtain it (including the treatment of the partial presence of vehicles on the link).
2. Determine the FFS of the study site, either from field data or by estimating it according to the Chapter 11 method for basic freeway segments.
3. Enter all available input characteristics (both geometric and traffic characteristics) into the simulator. The length of the segment or link to be simulated should be 1,500 ft, to correspond to the HCM-defined area of influence. Install virtual detectors within the area of influence and at the downstream end of the study segment to obtain density, speeds, and flows.
4. Load the study network above capacity to obtain the maximum throughput, and compare the result with the HCM estimate. Calibrate the simulator by modifying parameters related to the minimum time headway so that the simulated capacity matches the HCM estimate. Estimate the required number of simulation runs that will need to be conducted to produce a statistically valid comparison.
5. Compare the flow approaching the two rightmost lanes with the HCM's estimate. Adjust the simulation parameters related to driver awareness of upcoming turns to match the HCM-predicted v_{12} value.

Example Problems Illustrating Alternative Tool Applications

Chapter 28, Freeway Merges and Diverges: Supplemental, includes two example problems that examine situations beyond the scope of this chapter's methodology by using a typical microsimulation-based tool. Both problems are based on this chapter's Example Problem 3, which analyzes an eight-lane freeway segment with an entrance and an exit ramp. The first problem evaluates the effects of the addition of ramp metering, while the second evaluates the impacts of converting the leftmost lane of the mainline into an HOV lane.

4. EXAMPLE PROBLEMS

Exhibit 13-21
List of Example Problems

Example Problem	Title	Type of Analysis
1	Isolated One-Lane, Right-Hand On-Ramp to a Four-Lane Freeway	Operational analysis
2	Two Adjacent Single-Lane, Right-Hand Off-Ramps on a Six-Lane Freeway	Operational analysis
3	One-Lane On-Ramp Followed by a One-Lane Off-Ramp on an Eight-Lane Freeway	Operational analysis
4	Single-Lane, Left-Hand On-Ramp on a Six-Lane Freeway	Special case
5	Service Flow Rates and Service Volumes for an Isolated On-Ramp on a Six-Lane Freeway	Service flow rates and service volumes

EXAMPLE PROBLEM 1: ISOLATED ONE-LANE, RIGHT-HAND ON-RAMP TO A FOUR-LANE FREEWAY

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Isolated location (no adjacent ramps to consider)
- One-lane ramp roadway and junction
- Four-lane freeway (two lanes in each direction)
- Upstream freeway demand volume = 2,500 veh/h
- Ramp demand volume = 550 veh/h
- 10% trucks, 0% RVs on the freeway
- 5% trucks, 0% RVs on the ramp
- Acceleration lane = 740 ft
- FFS, freeway = 60 mi/h
- FFS, ramp = 45 mi/h
- Level terrain for freeway and ramp
- Peak hour factor = 0.90
- Drivers are regular commuters

Comments

All input parameters are known, so no default values are needed or used. Adjustment factors for heavy vehicles and driver population are found in Chapter 11, Basic Freeway Segments.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

Demand volumes are given for the freeway and the ramp. The PHF is specified. The driver population factor for commuters is 1.00 (Chapter 11), while the heavy vehicle adjustment factor is computed as follows:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Truck and RV presence is given. The value of E_T for level terrain is 1.5 (Chapter 11). On the basis of these values, the freeway and ramp demand volumes are converted as follows:

For the freeway:

$$f_{HV} = \frac{1}{1 + 0.10\,(1.5 - 1)} = 0.952$$

$$v_F = \frac{2{,}500}{0.90 \times 0.952 \times 1.00} = 2{,}918 \text{ pc/h}$$

For the ramp:

$$f_{HV} = \frac{1}{1 + 0.05\,(1.5 - 1)} = 0.976$$

$$v_F = \frac{550}{0.90 \times 0.976 \times 1.00} = 626 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Ramp Influence Area with Equation 13-2 and Exhibit 13-6

$$v_{12} = v_F \times P_{FM}$$

The freeway flow rate was computed in Step 1. The value of P_{FM} is found in Exhibit 13-6. For a four-lane freeway, the value is 1.00. Then

$$v_{12} = 2{,}918 \times 1.00 = 2{,}918 \text{ pc/h}$$

Because there are no outer lanes on a four-lane freeway, there is no need to check this result for reasonableness.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

The critical capacity checkpoint for a single-lane on-ramp is the downstream freeway segment:

$$v_{FO} = v_F + v_R = 2{,}918 + 626 = 3{,}544 \text{ pc/h}$$

The capacity of a four-lane freeway (two lanes in one direction) with an FFS of 60 mi/h is given in Exhibit 13-8. The capacity is 4,600 pc/h, which is more than the demand flow of 3,544 pc/h. The capacity of a one-lane ramp with an FFS of 45 mi/h is given in Exhibit 13-10 as 2,100 pc/h, which is well in excess of the ramp demand flow of 626 pc/h. The maximum desirable flow rate entering the ramp influence area is also 4,600 pc/h, again more than 3,544. Thus, the operation of the segment is expected to be stable. LOS F does not exist.

Step 4: Compute Density and Find LOS by Using Equation 13-21 and Exhibit 13-2

The estimated density in the ramp–freeway junction is estimated by using Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

$$D_R = 5.475 + (0.00734 \times 626) + (0.0078 \times 2{,}918) - (0.00627 \times 740) = 28.2 \text{ pc/mi/ln}$$

From Exhibit 13-2, this is LOS D, but the result is close to the LOS C boundary.

Step 5: Compute Merge Area Speed as Supplemental Information by Using Exhibit 13-11

Since there are no outer lanes present on a four-lane freeway, only the speed within the ramp influence area should be computed:

$$S_R = FFS - (FFS - 42) M_S$$

$$M_S = 0.321 + 0.0039 e^{(v_{R12}/1{,}000)} - 0.002\left(L_A S_{FR} / 1{,}000\right)$$

$$M_S = 0.321 + 0.0039 e^{(3{,}544/1{,}000)} - 0.002(740 \times 45 / 1{,}000) = 0.389$$

$$S_R = 60 - (60 - 42) \times 0.389 = 53.0 \text{ mi/h}$$

Discussion

The results indicate that the merge area operates in a stable fashion, with some deterioration in density and speed due to merging operations.

EXAMPLE PROBLEM 2: TWO ADJACENT SINGLE-LANE, RIGHT-HAND OFF-RAMPS ON A SIX-LANE FREEWAY

The Facts

The following information concerning demand volumes and geometries is available for this problem:

- Two consecutive one-lane, right-hand off-ramps
- Six-lane freeway with FFS = 60 mi/h
- Rolling terrain for freeway and both ramps
- 5% trucks on freeway and both ramps; 0% RVs
- First ramp FFS = 40 mi/h
- Second ramp FFS = 25 mi/h
- Drivers are regular commuters
- Freeway demand volume = 4,500 veh/h (immediately upstream of the first off-ramp)
- First ramp demand volume = 300 veh/h
- Second ramp demand volume = 500 veh/h
- Distance between ramps = 750 ft
- First ramp deceleration lane length = 500 ft

- Second ramp deceleration lane length = 300 ft
- Peak hour factor = 0.95

Comments

The solution will use adjustment factors for heavy vehicle presence and driver population selected from Chapter 11, Basic Freeway Segments. All input parameters are specified, so no default values are needed or used.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

In this case, three demand volumes must be converted: the freeway volume immediately upstream of the first ramp and the two ramp demand volumes. Since all demands include 5% trucks and no RVs, only a single heavy vehicle adjustment factor will be needed. From Chapter 11, the appropriate value of E_T for rolling terrain is 2.5. For drivers who are regular commuters, the appropriate value of f_p is 1.00.

Then

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1)} = 0.930$$

and

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

$$v_F = \frac{4{,}500}{0.95 \times 0.930 \times 1.00} = 5{,}093 \text{ pc/h}$$

$$v_{R1} = \frac{300}{0.95 \times 0.930 \times 1.00} = 340 \text{ pc/h}$$

$$v_{R2} = \frac{500}{0.95 \times 0.930 \times 1.00} = 566 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Two Ramp Influence Areas by Using Equation 13-13 and Exhibit 13-7

Because there are two consecutive off-ramps under consideration, the first will have to consider the impact of the second on its operations, and the second will have to consider the impact of the first.

First Off-Ramp

From Exhibit 13-7, flow in Lanes 1 and 2 of the freeway is estimated by using Equation 13-11 or Equation 13-9, depending on whether the impact of the downstream off-ramp is significant. This is determined by computing the equivalence distance by using Equation 13-13:

$$L_{EQ} = \frac{v_D}{1.15 + 0.000032 v_F - 0.000369 v_{R1}}$$

$$L_{EQ} = \frac{566}{1.15 + (0.000032 \times 5{,}093) - (0.000369 \times 340)} = 657 \text{ ft}$$

Since the actual distance between ramps, 750 ft, is greater than the equivalence distance of 657 ft, the ramp may be treated as if it were isolated, with Equation 13-9:

$$P_{FD} = 0.760 - 0.000025 v_F - 0.000046 v_{R1}$$

$$P_{FD} = 0.760 - (0.000025 \times 5093) - (0.000046 \times 340) = 0.617$$

Then

$$v_{12} = v_R + (v_F - v_R) P_{FD}$$

$$v_{12} = 340 + (5{,}093 - 340) \times 0.617 = 3{,}273 \text{ pc/h}$$

Because a six-lane freeway includes one outer lane (Lane 3), the reasonableness of the predicted lane distribution of arriving freeway vehicles should be checked. The flow rate in Lane 3 is 5,093 – 3,273 = 1,820 pc/h. The average flow per lane in Lanes 1 and 2 is 3,273/2 = 1,637 pc/h (rounded to the nearest pc). Then:

Is v_3 > 2,700 pc/h/ln?	**No**
Is v_3 > 1.5 × (1,637) = 2,456 pc/h/ln?	**No**

Since both checks for reasonable lane distribution are passed, the computed value of v_{12} for the first off-ramp is accepted as 3,273 pc/h.

Second Off-Ramp

From Exhibit 13-7, the second off-ramp should be analyzed by using Equation 13-9, which is for an isolated off-ramp. Adjacent upstream off-ramps do not affect the lane distribution of arriving vehicles at a downstream off-ramp.

The freeway flow approaching Ramp 2, however, includes the freeway flow approaching Ramp 1, less the flow rate of vehicles exiting the freeway at Ramp 1. Therefore, the freeway flow rate approaching Ramp 2 is as follows:

$$v_{F2} = 5{,}093 - 340 = 4{,}753 \text{ pc/h}$$

Then

$$P_{FD} = 0.760 - (0.000025 \times 4753) - (0.000046 \times 566) = 0.615$$

$$v_{12} = 566 + (4{,}753 - 566) \times 0.615 = 3{,}141 \text{ pc/h}$$

Again, because there is an outer lane on a six-lane freeway, the reasonableness of this estimate must be checked. The flow rate in the outer lane v_3 is 4,753 – 3,141 = 1,612 pc/h. The average flow rate in Lanes 1 and 2 is 3,141/2 = 1,571 pc/h (rounded). Then:

Is $v_3 > 2{,}700$ pc/h/ln? **No**

Is $v_3 > 1.5 \times 1{,}571 = 2{,}357$ pc/h/ln? **No**

Once again, the predicted lane distribution of arriving vehicles is reasonable, and v_{12} is taken to be 3,141 pc/h.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

Because two off-ramps are involved in this segment, there are several capacity checkpoints:

- Total freeway flow upstream of the first off-ramp (the point at which maximum freeway flow exists),
- Capacity of both off-ramps, and
- Maximum desirable flow rates entering each of the two off-ramp influence areas.

These comparisons are shown in Exhibit 13-22. Note that freeway capacity is based on a freeway with FFS = 60 mi/h. The first ramp capacity is based on a ramp FFS of 40 mi/h and the second on a ramp FFS of 25 mi/h.

Exhibit 13-22
Capacity Checks for Example Problem 2

Item	Capacity (pc/h) Exhibit 13-8, Exhibit 13-10	Demand Flow Rate (pc/h)	Problem?
Freeway flow rate	6,900	5,093	No
First off-ramp	2,000	340	No
Second off-ramp	1,900	566	No
Max. v_{12} first ramp	4,400	3,373	No
Max. v_{12} second ramp	4,400	3,141	No

None of the capacity values are exceeded, so operation of these ramp junctions will be stable, and LOS F does not occur.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-22 and Exhibit 13-2

Because there are two off-ramps, two ramp influence areas are involved, and two ramp influence area densities will be computed.

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

$$D_{R1} = 4.252 + (0.0086 \times 3{,}273) - (0.009 \times 500) = 27.9 \text{ pc/mi/ln}$$

$$D_{R2} = 4.252 + (0.0086 \times 3{,}141) - (0.009 \times 300) = 28.6 \text{ pc/mi/ln}$$

From Exhibit 13-2, both of these ramp influence areas operate very close to the boundary between LOS C and LOS D (28.0 pc/mi/ln). Ramp 1 operates in LOS C, while Ramp 2 operates in LOS D.

While it makes virtually no difference in this case, note that the two ramp influence areas overlap. The influence area of the first off-ramp extends 1,500 ft upstream. The influence area of the second off-ramp also extends 1,500 ft

upstream. Given that the ramps are only 750 ft apart, the second ramp influence area overlaps the first for 750 ft (immediately upstream of the first diverge point). Normally, the worst of the two levels of service would be applied to this 750-ft overlap. In this case, the levels of service are the same. Indeed, the predicted densities are virtually equal, so the impact of the overlap is minimal, and the predicted values are not really affected.

Step 5: Compute Diverge Area Speeds as Supplemental Information by Using Exhibit 13-12 and Exhibit 13-13

Because these ramps are on a six-lane freeway with an outer lane, it is possible to estimate the speed within each ramp influence area, the speed in the outer lane adjacent to each ramp influence area, and the weighted average of the two.

First Off-Ramp

The speed within the first ramp influence area is computed as follows:

$$D_S = 0.883 + 0.00009 v_R - 0.013 S_{FR}$$

$$D_S = 0.883 + (0.00009 \times 3{,}273) - (0.013 \times 40) = 0.394$$

$$S_R = FFS - (FFS - 42) D_S = 60 - (60 - 42) \times 0.394 = 52.9 \text{ mi/h}$$

The flow rate in the outer lane (v_{OA}) is 5,093 – 3,273 = 1,820 pc/h/ln. The average speed in this outer lane is computed as follows:

$$S_O = 1.097 FFS - 0.0039\,(v_{OA} - 1{,}000)$$

$$S_O = (1.097 \times 60) - 0.0039 \times (1{,}820 - 1{,}000) = 62.6 \text{ mi/h}$$

The average speed in Lane 3 is predicted to be slightly higher than the FFS of the freeway. This is not uncommon, since through vehicles at higher speeds use Lane 3 to avoid congestion in the ramp influence area. The average speed across all lanes, however, should not be higher than the FFS. In this case, the average speed across all lanes is computed as follows:

$$S = \frac{3{,}273 + (1{,}820 \times 1)}{\left(\frac{3{,}273}{52.9}\right) + \left(\frac{1{,}820 \times 1}{62.0}\right)} = 56.0 \text{ mi/h}$$

This result is, as expected, less than the FFS of the freeway.

Second Off-Ramp

The speed in the second ramp influence area is computed as follows:

$$D_S = 0.883 + (0.00009 \times 566) - (0.013 \times 25) = 0.609$$

$$S_R = 60 - (60 - 42) \times 0.609 = 49.0 \text{ mi/h}$$

Lane 3 has a demand flow rate of 4,753 – 3,141 = 1,612 pc/h/ln. The average speed in this outer lane is computed as follows:

$$S_O = (1.097 \times 60) - 0.0039 \times (1{,}612 - 1{,}000) = 63.4 \text{ mi/h}$$

The average speed across all freeway lanes is

$$S = \frac{3{,}141 + (1{,}612 \times 1)}{\left(\frac{3{,}141}{49.0}\right) + \left(\frac{1{,}612 \times 1}{63.4}\right)} = 53.1 \text{ mi/h}$$

Discussion

The speed results in this case are interesting. While densities are similar for both ramps, the density is somewhat higher and the speed somewhat lower in the second influence area. This is primarily the result of a shorter deceleration lane and a lower ramp FFS (25 mi/h versus 40 mi/h). In both cases, the average speed in the outer lane is higher than the FFS, which applies as an average across all lanes.

Since the operation is stable, there is no special concern here, short of a significant increase in demand flows. LOS is technically D but falls just over the LOS C boundary. This is a case in which the step-function LOS assigned may imply an operation poorer than actually exists. It emphasizes the importance of knowing not only the LOS but also the value of the service measure that produces it.

EXAMPLE PROBLEM 3: ONE-LANE ON-RAMP FOLLOWED BY A ONE-LANE OFF-RAMP ON AN EIGHT-LANE FREEWAY

The Facts

The following information is available concerning this pair of ramps to be analyzed:

- Eight-lane freeway with an FFS of 65 mi/h
- One-lane, right-hand on-ramp with an FFS of 30 mi/h
- One-lane, right-hand off-ramp with an FFS of 25 mi/h
- Distance between ramps = 1,300 ft
- Acceleration lane on Ramp 1 = 260 ft
- Deceleration lane on Ramp 2 = 260 ft
- Level terrain on freeway and both ramps
- 10% trucks, no RVs on freeway and off-ramp
- 5% trucks, no RVs on on-ramp
- Freeway flow rate (upstream of first ramp) = 5,500 veh/h
- On-ramp flow rate = 400 veh/h
- Off-ramp flow rate = 600 veh/h
- PHF = 0.90
- Drivers are regular commuters

Comments

As with previous example problems, the conversion of demand volumes to flow rates requires adjustment factors selected from Chapter 11, Basic Freeway

Segments. All pertinent information is given, and no default values will be applied.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

Three demand volumes must be converted to flow rates under equivalent ideal conditions: the freeway volume immediately upstream of the first ramp junction, the first ramp volume, and the second ramp volume. Because the freeway segment under study has level terrain, the value of E_T will be 1.5 for all volumes. Because the drivers are regular commuters, the driver population factor, f_p, is 1.00.

Then, for the freeway demand volume:

$$f_{HV} = \frac{1}{1+0.10(1.5-1)} = 0.952$$

$$v_F = \frac{5{,}500}{0.90 \times 0.952 \times 1.00} = 6{,}419 \text{ pc/h}$$

For the on-ramp demand volume:

$$f_{HV} = \frac{1}{1+0.05(1.5-1)} = 0.976$$

$$v_{R1} = \frac{400}{0.90 \times 0.976 \times 1.00} = 455 \text{ pc/h}$$

For the off-ramp demand volume:

$$f_{HV} = \frac{1}{1+0.10(1.5-1)} = 0.952$$

$$v_{R2} = \frac{600}{0.90 \times 0.952 \times 1.00} = 700 \text{ pc/h}$$

In the remaining computations, these converted demand flow rates are used as input values.

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Two Ramp Influence Areas by Using Equation 13-2 and Exhibit 13-6 for the On-Ramp and Equation 13-8 and Exhibit 13-7 for the Off-Ramp

Once again, the situation involves a pair of adjacent ramps. Each ramp must consider the potential impact of the other on its operations. Because the ramps are on an eight-lane freeway (four lanes in each direction), Exhibit 13-6 and Exhibit 13-7 indicate that each ramp is considered as if it were isolated.

First Ramp (On-Ramp)

Equation 13-2 and Exhibit 13-6 apply to on-ramps. Exhibit 13-6 presents two possible equations for use in estimating v_{12} on the basis of the value of v_F/S_{FR}. In this case, the value is 6,419/30 = 210.6 > 72. Therefore, Equation 13-5 is used, giving the following:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.2178 - 0.000125 v_R$$

$$P_{FM} = 0.2178 - (0.000125 \times 455) = 0.161$$

$$v_{12} = 6{,}419 \times 0.161 = 1{,}033 \text{ pc/h}$$

Because the eight-lane freeway includes two outer lanes in each direction, the reasonableness of this prediction must be checked. The average flow per lane in Lanes 1 and 2 is 1,033/2 = 517 pc/h/ln (rounded). The flow in the two outer lanes, Lanes 3 and 4, is 6,419 – 1,033 = 5,386 pc/h. The average flow per lane in Lanes 3 and 4 is, therefore, 5,386/2 = 2,693 pc/h/ln. Then:

Is v_{av34} > 2,700 pc/h/ln? **No**

Is v_{av34} > 1.5 × 517 = 776 pc/h/ln? **Yes**

The predicted lane distribution, therefore, is not reasonable. Too many vehicles are placed in the two outer lanes compared with Lanes 1 and 2. Equation 13-19 is used to produce a more reasonable distribution:

$$v_{12a} = \left(\frac{v_F}{2.50}\right) = \left(\frac{6{,}419}{2.50}\right) = 2{,}568 \text{ pc/h}$$

On the basis of this adjusted value, the number of vehicles now assigned to the two outer lanes is 6,419 – 2,568 = 3,851 pc/h.

Second Ramp (Off-Ramp)

Equation 13-8 and Exhibit 13-7 apply to off-ramps. Exhibit 13-7 shows that the value of P_{FD} for off-ramps on eight-lane freeways is a constant: 0.436. As the methodology is based on regression analysis of a database, the recommendation of a constant reflects a small sample size in that database. Note also that the freeway flow approaching the second ramp is the sum of the freeway flow approaching the first ramp and the on-ramp flow that is now also on the freeway, or 6,419 + 455 = 6,874 pc/h. The flow rate in Lanes 1 and 2 is now easily computed by using Equation 13-8:

$$v_{12} = v_R + (v_F - v_R) P_{FD}$$

$$v_{12} = 700 + (6{,}874 - 700) \times 0.436 = 3{,}392 \text{ pc/h}$$

Because there are two outer lanes on this eight-lane freeway, the reasonableness of this estimate must be checked. The average flow per lane in Lanes 1 and 2 is 3,392/2 = 1,696 pc/h/ln. The total flow in Lanes 3 and 4 of the freeway is 6,874 – 3,392 = 3,482 pc/h, or an average flow rate per lane of 3,482/2 = 1,741 pc/h/ln.

Is v_{av34} > 2,700 pc/h/ln? **No**

Is $v_{av34} > 1.5 \times 1{,}696 = 2{,}544$ pc/h/ln? **No**

Therefore, the estimated value of v_{12} is deemed reasonable and is carried forward in the computations.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

Because there are two ramps in this segment, there are five capacity checkpoints to consider:

- The freeway flow rate at its maximum point—which in this case is between the on- and off-ramp, since this is the only location where both on- and off-ramp vehicles are on the freeway.
- The capacity of the on-ramp.
- The capacity of the off-ramp.
- The maximum desirable flow entering the on-ramp influence area.
- The maximum desirable flow entering the off-ramp influence area.

These comparisons are shown in Exhibit 13-23. The capacity of the freeway is based on an eight-lane freeway with an FFS of 65 mi/h. The capacity of the on-ramp is based on an FFS of 30 mi/h, and the capacity of the off-ramp is based on an FFS of 25 mi/h.

Exhibit 13-23
Capacity Checks for Example Problem 3

Item	Capacity (pc/h) Exhibit 13-8, Exhibit 13-10	Demand Flow Rate (pc/h)	Problem?
Freeway flow rate	9,400	6,874	No
First on-ramp	1,900	345	No
Second off-ramp	1,900	700	No
Max. v_{R12} first ramp	4,600	2,568 + 455 = 3,023	No
Max. v_{12} second ramp	4,400	3,392	No

There are no capacity concerns, since all demands are well below the associated capacities or maximum desirable values. LOS F is not present in any part of this segment, and operations are expected to be stable.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-21, Equation 13-22, and Exhibit 13-2

Equation 13-21 is used to find the density in the first on-ramp influence area:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

$$D_R = 5.475 + (0.00734 \times 455) + (0.0078 \times 2{,}568) - (0.00627 \times 260) = 27.2 \text{ pc/mi/ln}$$

Equation 13-22 is used to find the density in the second off-ramp influence area:

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

$$D_R = 4.252 + (0.0086 \times 3{,}391) - (0.009 \times 260) = 31.1 \text{ pc/mi/ln}$$

From Exhibit 13-2, both of these ramp influence areas operate very close to the boundary between LOS C and LOS D (28 pc/mi/ln). Ramp 1 operates in LOS C, while Ramp 2 operates in LOS D.

Because the on-ramp influence area extends 1,500 ft downstream, the off-ramp influence area extends 1,500 ft upstream, and the two ramps are only 1,300 ft apart, the distance between the ramps is included in both. Therefore, the more pessimistic prediction of LOS D for the off-ramp governs the operation. Oddly, the additional 200 ft of the off-ramp influence area is actually upstream of the on-ramp, and the additional 200 ft of the on-ramp influence area is downstream of the off-ramp.

Step 5: Compute Merge and Diverge Area Speeds as Supplemental Information by Using Exhibit 13-11 and Exhibit 13-12

Because of the eight-lane freeway, speeds should be estimated for the two ramp influence areas, for the outer lanes (Lanes 3 and 4) adjacent to the ramp influence area, and for all vehicles—the weighted average of the other two speeds.

First Ramp (On-Ramp)

Equations for estimation of average speed in an on-ramp influence area and in outer lanes adjacent to it are taken from Exhibit 13-11.

$$M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002\left(L_A S_{FR}/1{,}000\right)$$

$$M_S = 0.321 + 0.0039e^{(3{,}032/1{,}000)} - 0.002(260/30) = 0.385$$

$$S_R = FFS - (FFS - 42)M_S = 65 - (65 - 42) \times 0.385 = 56.2 \text{ mi/h}$$

Since the average outer lane demand flow rate is 3,851/2 = 1,926 pc/h/ln, which is greater than 500 pc/h/ln and less than 2,300 pc/h/ln, the outer speed is estimated as follows:

$$S_O = FFS - 0.0036\left(v_{OA} - 500\right)$$

$$S_O = 65 - 0.0036(1{,}926 - 500) = 59.9 \text{ mi/h}$$

The weighted average speed of all vehicles is

$$S = \frac{3{,}032 + (1{,}926 \times 2)}{\left(\dfrac{3{,}032}{56.2}\right) + \left(\dfrac{1{,}926 \times 2}{59.9}\right)} = 58.2 \text{ mi/h}$$

Second Ramp (Off-Ramp)

For off-ramps, equations for estimation of average speed are drawn from Exhibit 13-12. At the second ramp, the flow in Lanes 1 and 2 has been computed as 3,392 pc/h or 1,696 pc/h/ln, while the flow in Lanes 3 and 4 is 3,482 pc/h, or 1,741 pc/h/ln. Then

$$D_S = 0.883 + 0.00009v_R - 0.013S_{FR}$$

$$D_S = 0.883 + (0.00009 \times 700) - (0.013 \times 25) = 0.621$$

$$S_R = FFS - (FFS - 42)D_S$$

$$S_R = 65 - (65 - 42) \times 0.621 = 50.7 \text{ mi/h}$$

Because the average flow in the outer lanes is greater than 1,000 pc/h/ln, the average speed of vehicles in the outer lanes (Lanes 3 and 4) is as follows:

$$S_O = 1.097FFS - 0.0039\left(v_{OA} - 1{,}000\right)$$

$$S_O = (1.097 \times 65) - 0.0039\left(1{,}741 - 1{,}000\right) = 68.4 \text{ mi/h}$$

The weighted average speed of all vehicles is

$$S = \frac{3{,}392 + \left(1{,}741 \times 2\right)}{\left(\dfrac{3{,}392}{50.7}\right) + \left(\dfrac{1{,}741 \times 2}{68.4}\right)} = 58.3 \text{ mi/h}$$

Discussion

As noted previously, between the ramps, the influence areas of both ramps fully overlap. Since a higher density is predicted for the off-ramp influence area, and LOS D results, this density should be applied to the entire area between the two ramps.

The speed results are also interesting. The slower speeds within the off-ramp influence area will also control the overlap area. On the other hand, the speed results indicate a higher average speed for all vehicles associated with the off-ramp than the speed associated with the on-ramp. This is primarily due to the much larger disparity between speeds within the ramp influence area and in outer lanes when the off-ramp is considered. The speed differential is more than 20 mi/h for the off-ramp, as opposed to a little more than 3 mi/h for the on-ramp. This is not entirely unexpected. At diverge junctions, vehicles in outer lanes tend to face less turbulence than those in outer lanes near merge junctions. All off-ramp vehicles must be in Lanes 1 and 2 for some distance before exiting the freeway. On-ramp vehicles, on the other hand, can execute as many lane changes as they wish—consistent with safety and sanity—and more of them may wind up in outer lanes within 1,500 ft of the junction point.

Thus, the total operation of this two-ramp segment is expected to be LOS D, with speeds of approximately 50 mi/h in Lanes 1 and 2 and approximately 70 mi/h in Lanes 3 and 4.

EXAMPLE PROBLEM 4: SINGLE-LANE, LEFT-HAND ON-RAMP ON A SIX-LANE FREEWAY

The Facts

- One-lane, left-side on-ramp on a six-lane freeway (three lanes in each direction)
- Freeway demand volume upstream of ramp = 4,000 veh/h
- On-ramp demand volume = 500 veh/h
- 15% trucks, no RVs on freeway
- 5% trucks, no RVs on ramp
- Freeway FFS = 65 mi/h
- Ramp FFS = 30 mi/h

- Acceleration lane = 820 ft
- Level terrain on freeway and ramp
- Drivers are regular commuters

Comments

This is a special application of the ramp analysis methodology presented in this chapter. For left-hand ramps, the flow rate in Lanes 1 and 2 (v_{12}) is initially computed as if it were a right-hand ramp. Exhibit 13-16 is then used to convert this result to an estimate of the flow in Lanes 2 and 3 (v_{23}), since these are the two leftmost lanes that will be involved in the merge. In effect, the ramp influence area is, in this case, Lanes 3 and 4 and the acceleration lane for a distance of 1,500 ft downstream of the merge point.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

From Chapter 11, Basic Freeway Segments, the passenger car equivalent E_T for trucks in level terrain is 1.5. The driver population adjustment factor f_p for regular commuters is 1.00.

For the freeway demand volume:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.15(1.5 - 1)} = 0.930$$

$$v_F = \frac{4{,}000}{0.90 \times 0.93 \times 1.00} = 4{,}779 \text{ pc/h}$$

For the ramp demand volume:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1)} = 0.976$$

$$v_R = \frac{500}{0.90 \times 0.976 \times 1.00} = 569 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 2 and 3 Immediately Upstream of the Ramp Influence Area by Using Equation 13-2 and Exhibit 13-6

To estimate flow in the two left lanes, the flow normally expected in Lanes 1 and 2 for a similar right-hand ramp must first be computed. From Exhibit 13-6, for an isolated on-ramp on a six-lane freeway, Equation 13-4 is used:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.5775 + 0.000028 L_A$$

$$P_{FM} = 0.5775 + (0.000028 \times 820) = 0.600$$

$$v_{12} = 4{,}779 \times 0.600 = 2{,}867 \text{ pc/h}$$

From Exhibit 13-16, the adjustment factor applied to this result to find the estimated flow rate in Lanes 2 and 3 is 1.12. Therefore:

$$v_{23} = 2{,}867 \times 1.12 = 3{,}211 \text{ pc/h}$$

While, strictly speaking, the reasonableness criteria for lane distribution do not apply to left-hand ramps, they can be applied very approximately. In this case, the single "outer lane" (which is now Lane 1) would have a flow rate of 4,779 – 3,211 = 1,568 pc/h. This is not greater than 2,700 pc/h/ln, nor is it greater than 1.5 times the average flow in Lanes 2 and 3 (1.5 × 3,211/2 = 2,408 pc/h/ln). Thus, even if the reasonableness criteria were approximately applied in this case, no violation would exist.

The remaining computations proceed for the left-hand ramp, with the substitution of v_{34} for v_{12} in all algorithms used.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

For this case, there are three simple checkpoints:

- The principal capacity checkpoint is the total demand flow rate downstream of the merge, 4,779 + 569 = 5,348 pc/h. From Exhibit 13-8, for a six-lane freeway with an FFS of 65 mi/h, the capacity is 7,050 pc/h, well over the demand flow rate.
- The ramp roadway capacity should also be checked by using Exhibit 13-10. For a single-lane ramp with an FFS of 30 mi/h, the capacity is 1,900 pc/h, which is much greater than the demand flow rate of 569 pc/h.
- Finally, the maximum flow entering the ramp influence area should be checked. In this case, a left-hand ramp, the total flow entering the ramp influence area is the freeway flow remaining in Lanes 2 and 3 plus the ramp flow rate. Thus, the total flow entering the ramp influence area is 3,211 + 569 = 3,780 pc/h, which is lower than the maximum desirable flow rate of 4,600 pc/h, shown in Exhibit 13-8.

Thus, there are no capacity problems at this merge point, and stable operations are expected. LOS F will not result from the stated conditions.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-21 and Exhibit 13-2

The density in the ramp influence area is found by using Equation 13-21, except v_{23} replaces v_{12} because of the left-hand ramp placement:

$$D_S = 5.475 + 0.00734 v_R + 0.0078 v_{23} - 0.00627 L_A$$

$$D_S = 5.475 + (0.00734 \times 569) + (0.0078 \times 3{,}211) - (0.00627 \times 820) = 29.6 \text{ pc/mi/ln}$$

From Exhibit 13-2, this is LOS D.

Step 5: Compute Merge and Diverge Area Speeds as Supplemental Information by Using Exhibit 13-11 and Exhibit 13-13

The speed estimation algorithms were calibrated for right-hand ramps, and the estimation algorithms for "outer lane(s)" assume that these are the leftmost lanes. Thus, for a left-hand ramp, these computations must be considered approximate at best.

By using the equations in Exhibit 13-11 and Exhibit 13-13, the following results are obtained:

$$M_S = 0.321 + 0.0039 e^{(3{,}780/1{,}000)} - 0.002(820 \times 30/1{,}000) = 0.443$$

$$S_R = 65 - (65 - 42) \times 0.443 = 54.8 \text{ mi/h}$$

$$S_O = 65 - 0.0036\ (1{,}568 - 500) = 61.2 \text{ mi/h}$$

$$S = \frac{3{,}780 + (1{,}568 \times 1)}{\left(\dfrac{3{,}780}{54.8}\right) + \left(\dfrac{1{,}568 \times 1}{61.2}\right)} = 56.5 \text{ mi/h}$$

While traffic in the outer lane is predicted to travel somewhat faster than traffic in the lanes in the ramp influence area (which includes the acceleration lane), the approximate nature of the speed result for left-hand ramps makes it difficult to draw any firm conclusions concerning speed behavior.

Discussion

This example problem is typical of the way the situations in the Special Cases section are treated. Modifications as specified are applied to the standard algorithms used for single-lane, right-hand ramp junctions. In this case, operations are acceptable, but in LOS D—though not far from the LOS C boundary. Because the left-hand lanes are expected to carry freeway traffic flowing faster than right-hand lanes, right-hand ramps are normally preferable to left-hand ramps when they can be provided without great difficulty.

EXAMPLE PROBLEM 5: SERVICE FLOW RATES AND SERVICE VOLUMES FOR AN ISOLATED ON-RAMP ON A SIX-LANE FREEWAY

The Facts

The following facts have been established for this situation:

- Single-lane, right-hand on-ramp with an FFS of 40 mi/h
- Six-lane freeway (three lanes in each direction) with an FFS of 70 mi/h
- Level terrain for freeway and ramp
- 12% trucks, 3% RVs on freeway
- 5% trucks, 2% RVs on ramp
- Peak hour factor = 0.87
- Drivers are regular users of the facility
- Acceleration lane = 1,000 ft

Comments

This example illustrates the computation of service flow rates and service volumes for a ramp–freeway junction. The case selected is relatively straightforward to avoid cluttering the illustration with extraneous complications that have been addressed in other example problems.

Two approaches will be demonstrated:

1. The ramp demand flow rate will be stated as a fixed percentage of the arriving freeway flow rate. The service flow rates and service volumes are expressed as arriving freeway flow rates that result in the threshold densities within the ramp influence area that define the limits of the various levels of service. For this computation, the ramp flow is set at 10% of the approaching freeway flow rate.
2. A fixed freeway demand flow rate will be stated, with service flow rates and service volumes expressed as ramp demand flow rates that result in the threshold densities within the ramp influence area that define the limits of the various levels of service. For this computation, the approaching freeway flow rate is set at 4,000 veh/h.

For LOS E, density does not define the limiting value of service flow rate, which is analogous to capacity for ramp–freeway junctions. It is defined as the flow that results in capacity being reached on the downstream freeway segment or ramp roadway.

Since all algorithms in this methodology are calibrated for passenger cars per hour under equivalent ideal conditions, initial computations are made in those terms. Results are then converted to service flow rates by using the appropriate heavy vehicle and driver population adjustment factors. Service flow rates are then converted to service volumes by multiplying by the peak hour factor.

From Exhibit 13-2, the following densities define the limits of LOS A–D:

LOS A	10 pc/mi/ln
LOS B	20 pc/mi/ln
LOS C	28 pc/mi/ln
LOS D	35 pc/mi/ln

From Exhibit 13-8 and Exhibit 13-10, capacity (or the threshold for LOS E) occurs when the downstream freeway flow rate reaches 7,200 pc/h (FFS = 70 mi/h) or when the ramp flow rate reaches 2,000 pc/h (ramp FFS = 40 mi/h).

Case 1: Ramp Demand Flow Rate = 0.10 Freeway Demand Flow Rate

Equation 13-21 defines the density in an on-ramp influence area as follows:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

In this case

v_R = 0.10 v_F

L_A = 1,000 ft

Equation 13-2 and Exhibit 13-6 give the following:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.5775 + 0.000028 L_A$$

$$P_{FM} = 0.5775 + (0.000028 \times 1{,}000) = 0.6055$$

$$v_{12} = 0.6055 v_F$$

Substitution of these values into Equation 13-21 gives

$$D_R = 5.475 + (0.00734 \times 0.10 v_F) + (0.0078 \times 0.6055 v_F) - (0.00627 \times 1{,}000)$$

$$D_R = 5.475 + 0.000734 v_F + 0.00472 v_F - 6.27$$

$$D_R = 0.005454 v_F - 0.795$$

$$v_F = \frac{D_R + 0.795}{0.005454}$$

This equation can now be solved for threshold values of v_F for LOS A through D by using the appropriate threshold values of density. The results will be in terms of service flow rates under equivalent ideal conditions:

$$v_F(\text{LOS A}) = \frac{10 + 0.795}{0.005454} = 1{,}979 \text{ pc/h}$$

$$v_F(\text{LOS B}) = \frac{20 + 0.795}{0.005454} = 3{,}813 \text{ pc/h}$$

$$v_F(\text{LOS C}) = \frac{28 + 0.795}{0.005454} = 5{,}280 \text{ pc/h}$$

$$v_F(\text{LOS D}) = \frac{35 + 0.795}{0.005454} = 6{,}563 \text{ pc/h}$$

At capacity, the limiting flow rate occurs when the downstream freeway segment is 7,200 pc/h. If the ramp flow rate is 0.10 of the approaching freeway flow rate, then

$$v_{FO} = 7{,}200 = v_F + 0.10 v_F = 1.10 v_F$$

$$v_F(\text{LOS E}) = \frac{7{,}200}{1.10} = 6{,}545 \text{ pc/h}$$

This must be checked to ensure that the ramp flow rate (0.10 × 6,545 = 655 pc/h) does not exceed the ramp capacity of 2,000 pc/h. Since it does not, the computation stands.

Note, however, that the LOS E (capacity) threshold is lower than the LOS D threshold. This indicates that LOS D operation cannot be achieved at this location. Before densities reach the 35 pc/h/ln threshold for LOS D, the capacity of the merge junction has been reached. Thus, there is no service flow rate or service volume for LOS D.

The computed values, as noted, are in terms of passenger cars per hour under equivalent ideal conditions. To convert these to service flow rates in vehicles per hour under prevailing conditions, they must be multiplied by the

heavy vehicle adjustment factor and the driver population factor. The approaching freeway flow includes 12% trucks and 3% RVs. For level terrain (Chapter 11, Basic Freeway Segments), $E_T = 1.5$ and $E_R = 1.2$. Then

$$f_{HV} = \frac{1}{1 + 0.12(1.5 - 1) + 0.03(1.2 - 1)} = 0.938$$

The driver population factor f_p for regular facility users is 1.00. Service volumes are obtained by multiplying service flow rates by the specified PHF, 0.87. These computations are illustrated in Exhibit 13-24.

Exhibit 13-24
Illustrative Service Flow Rates and Service Volumes Based on Approaching Freeway Demand

LOS	Service Flow Rate, Ideal Conditions (pc/h)	Service Flow Rate, Prevailing Conditions (SF) (veh/h)	Service Volume (SV) (veh/h)
A	1,979	1,979 × 0.938 × 1 = 1,856	1,856 × 0.87 = 1,615
B	3,813	3,813 × 0.938 × 1 = 3,577	3,577 × 0.87 = 3,112
C	5,280	5,280 × 0.938 × 1 = 4,953	4,953 × 0.87 = 4,309
D	NA	NA	NA
E	6,545	6,545 × 0.938 × 1 = 6,139	6,139 × 0.87 = 5,341

The service flow rates and service volumes shown in Exhibit 13-24 are stated in terms of the approaching freeway demand.

Case 2: Approaching Freeway Demand Volume = 4,000 veh/h

In this case, the approaching freeway demand will be held constant, and service flow rates and service volumes will be stated in terms of the ramp demand that can be accommodated at each LOS.

Since the freeway demand is stated in terms of an hourly volume in mixed vehicles per hour, it will be converted to passenger cars per hour under equivalent ideal conditions for use in the algorithms of this methodology:

$$v_F = \frac{V_F}{PHF \times f_{HV} \times f_p} = \frac{4{,}000}{0.87 \times 0.938 \times 1} = 4{,}902 \text{ pc/h}$$

The density is estimated by using Equation 13-20, and the variable P_{FM}—which is not dependent on v_R—remains 0.6055 as in Case 1. With a fixed value of freeway demand:

$$v_{12} = 0.6055 \times 4{,}902 = 2{,}968 \text{ pc/h}$$

Then, by using Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + (0.0078 \times 2{,}968) - (0.00627 \times 1{,}000)$$

$$D_R = 22.355 + 0.00734 v_R$$

$$v_R = \frac{D_R - 22.355}{0.00734}$$

It is apparent from this equation that neither LOS A (D_R = 10 pc/mi/ln) nor LOS B (D_R = 20 pc/mi/ln) can be achieved with a fixed freeway demand flow of 4,902 pc/h.

For LOS C and D:

$$v_R(\text{LOS C}) = \frac{28 - 22.355}{0.00734} = 769 \text{ pc/h}$$

$$v_R(\text{LOS D}) = \frac{35 - 22.355}{0.00734} = 1{,}723 \text{ pc/h}$$

Capacity, the limit of LOS E, occurs when the downstream freeway flow reaches 7,200 pc/h. With a fixed freeway demand:

$$v_{FO} = 7{,}200 = 4{,}902 + v_R$$

$$v_R(\text{LOS E}) = 7{,}200 - 4{,}902 = 2{,}298 \text{ pc/h}$$

This, however, violates the capacity of the ramp roadway, which is 2,000 pc/h. Thus, the limiting ramp flow rate for LOS E is set at 2,000 pc/h.

As in Case 1, these values are all stated in terms of passenger cars per hour under equivalent ideal conditions. They are converted to service flow rates by multiplying by the appropriate heavy vehicle and driver population adjustment factors. Since the ramp has a composition different from that of the approaching freeway flow, its adjustment must be recomputed:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1) + 0.02(1.2 - 1)} = 0.972$$

Service flow rates are converted to service volumes by multiplying by the peak hour factor. These computations are illustrated in Exhibit 13-25.

Exhibit 13-25
Illustrative Service Flow Rates and Service Volumes Based on a Fixed Freeway Demand

LOS	Service Flow Rate, Ideal Conditions (pc/h)	Service Flow Rate, Prevailing Conditions (SF) (veh/h)	Service Volume (SV) (veh/h)
A	NA	NA	NA
B	NA	NA	NA
C	769	769 × 0.972 × 1 = 747	747 × 0.87 = 650
D	1,723	1,723 × 0.972 × 1 = 1,675	1,675 × 0.87 = 1,457
E	2,000	2,000 × 0.972 × 1 = 1,944	1,944 × 0.87 = 1,691

These service flow rates and service volumes are based on a constant upstream arriving freeway demand and are stated in terms of limiting on-ramp demands for that condition.

Discussion

As this illustration shows, many considerations are involved in estimating service flow rates and service volumes for ramp–freeway junctions, not the least of which is specifying how such values should be defined. The concept of service flow rates and service volumes at specific ramp–freeway junctions is of limited utility. Since many of the details that affect the estimates will not be determined until final designs are prepared, operational analysis of the proposed design may be more appropriate.

Case 2 could have applications in considering how to time ramp meters. Appropriate limiting ramp flows can be estimated by using the same approach as for service volumes and service flow rates.

5. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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6. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses.* Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 14
MULTILANE HIGHWAYS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 14, Multilane Highways, addresses capacity and level-of-service (LOS) analysis for uninterrupted-flow segments of surface multilane highways. In general, uninterrupted flow may exist on a multilane highway if there are 2 mi or more between traffic signals. Where signals are more closely spaced, the facility should be analyzed as an urban street.

Many multilane highways will have periodic signalized intersections, even if the average signal spacing is well over 2 mi. In such cases, the multilane highway segments that are more than 2 mi away from any signalized intersections are analyzed by using the methodology of this chapter. Isolated signalized intersections should be analyzed with the methodology of Chapter 18, Signalized Intersections.

LOS procedures are provided for both automobiles and bicycles. The automobile methodology is based on the results of NCHRP Project 3-33 (*1*), and bicycle LOS is based on research conducted for the Florida Department of Transportation (2). The same methodology for bicycle LOS is used for both multilane and two-lane highways; readers interested in details of the bicycle methodology should refer to Chapter 15, Two-Lane Highways.

TYPES OF MULTILANE HIGHWAYS

Multilane highways generally have four to six lanes (in both directions) and posted speed limits between 40 and 55 mi/h. In some states, speed limits of 60 or 65 mi/h are used on some multilane highways. These highways may be divided by one of various median types, may be undivided (with only a centerline separating the directions of flow), or may have a two-way left-turn lane (TWLTL). They are typically located in suburban areas, leading into central cities, or along high-volume rural corridors, connecting two cities or two activity centers that generate a substantial number of daily trips. Exhibit 14-1 illustrates common types of multilane highways.

Traffic volumes on multilane highways vary widely but often have demand in the range of 15,000 to 40,000 veh/day. In some cases, volumes as high as 100,000 veh/day have been observed when access across the median is restricted and when major crossings are grade-separated. Bicycles are typically permitted on multilane highways, and multilane highways often serve as primary routes for both commuter cyclists (on suburban highways) and recreational cyclists (on rural highways).

BASE CONDITIONS

Base conditions include good weather, good visibility, and no incidents or accidents. These conditions are always assumed to exist.

The base conditions under which the full capacity of a multilane highway segment is achieved include good weather, good visibility, no incidents or accidents, no work zone activity, and no pavement defects that would affect operations. This chapter's methodology assumes that these conditions exist. If any of these conditions do not exist, the speed, LOS, and capacity of the multilane highway segment can be expected to be worse than the predictions by this methodology.

Exhibit 14-1
Multilane Highways

(a) Divided suburban multilane highway

(b) Undivided suburban multilane highway

(c) Suburban multilane highway with TWLTL

(d) Undivided rural multilane highway

Base conditions include the following conditions; the methodology can be adjusted to address situations in which these conditions do not exist:

Base conditions include 0% heavy vehicles and a driver population composed of regular users of the highway.

The methodology provides adjustments for situations in which these conditions do not apply.

- No heavy vehicles, such as trucks, buses, and recreational vehicles (RVs), in the traffic stream; and
- A driver population composed primarily of regular users who are familiar with the facility.

Characteristics such as lane width, total lateral clearance (TLC), median type, and access-point density will have an impact on the free-flow speed (FFS) of the facility. Curves describing operations under base conditions, however, account for differing FFSs.

FLOW CHARACTERISTICS UNDER BASE CONDITIONS

More severe geometric characteristics and the existence of access points are two key differences that result in lower multilane highway speeds and capacities than those of freeways with similar cross sections.

Uninterrupted flow on multilane highways is in most ways similar to that on basic freeway segments (Chapter 11). Several factors are different, however. Because side frictions are present in varying degrees from uncontrolled driveways and intersections as well as from opposing flows on undivided cross sections, speeds on multilane highways tend to be lower than those on similar basic freeway segments. The basic geometry of multilane highways also tends to be more severe than that of basic freeway segments because of the lower speed expectations. Last, isolated signalized intersections can exist along multilane highways. The overall result is that speeds and capacities on multilane highways are lower than those on basic freeway segments with similar cross sections.

Exhibit 14-2 shows speed–flow characteristics of multilane highway segments for various FFSs. Equations describing these curves are shown in Exhibit 14-3.

Curves are shown for FFSs between 45 mi/h and 60 mi/h. Because FFSs can vary widely, it is recommended that the FFS of a multilane highway segment be estimated to the nearest 5 mi/h, as follows:

The FFS of a multilane highway segment should be rounded to the nearest 5 mi/h.

42.5 mi/h ≤ FFS < 47.5 mi/h: use FFS = 45 mi/h,

47.5 mi/h ≤ FFS < 52.5 mi/h: use FFS = 50 mi/h,

52.5 mi/h ≤ FFS < 57.5 mi/h: use FFS = 55 mi/h,

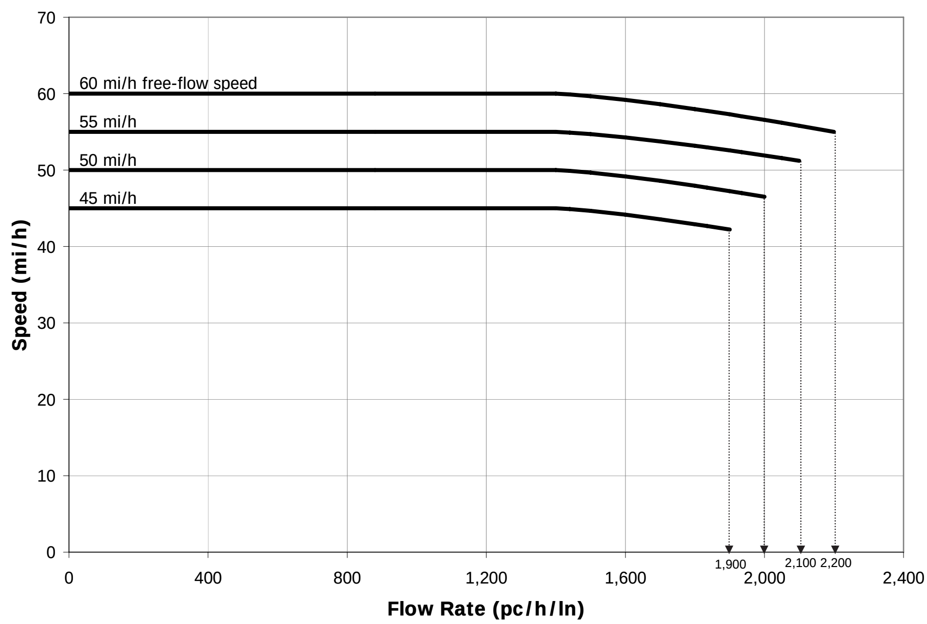
57.5 mi/h ≤ FFS < 62.5 mi/h: use FFS = 60 mi/h.

For multilane highway segments, speeds remain constant until they reach 1,400 pc/h/ln, after which speeds decline with further increases in flow rate.

Flow rates over 1,400 pc/h/ln result in speeds below the highway's FFS.

Exhibit 14-2
Speed–Flow Curves for Multilane Highways Under Base Conditions

LIVE GRAPH
Click here to view



Note: Maximum densities for LOS E occur at a *v/c* ratio of 1.00. These are 40, 41, 43, and 45 pc/mi/ln for FFSs of 60, 55, 50, and 45 mi/h, respectively.

Exhibit 14-3
Equations Describing Speed–Flow Curves in Exhibit 14-2

FFS (mi/h)	For $v_p \le$ 1,400 pc/h/ln, S (mi/h)	For $v_p >$ 1,400 pc/h/ln, S (mi/h)
60	60	$60-\left[5.00\times\left(\frac{v_p-1400}{800}\right)^{1.31}\right]$
55	55	$55-\left[3.78\times\left(\frac{v_p-1400}{700}\right)^{1.31}\right]$
50	50	$50-\left[3.49\times\left(\frac{v_p-1400}{600}\right)^{1.31}\right]$
45	45	$45-\left[2.78\times\left(\frac{v_p-1400}{500}\right)^{1.31}\right]$

Multilane highways with higher FFSs will also have higher base capacities. As most highways do not operate under base conditions, observed capacities will usually be lower than the base capacity.

Capacities represent an average flow rate across all lanes. Individual lanes could have higher stable flows.

CAPACITY OF MULTILANE HIGHWAY SEGMENTS

The capacity of a multilane highway segment under base conditions varies with the FFS. For 60-mi/h FFS, the capacity is 2,200 pc/h/ln. For lesser FFSs, capacity diminishes. For 55-mi/h FFS, the capacity is 2,100 pc/h/ln; for 50-mi/h FFS, 2,000 pc/h/ln; and for 45-mi/h FFS, 1,900 pc/h/ln.

These values represent national norms. Capacity varies stochastically, and any given location could have a larger or smaller value. In addition, capacity refers to the average flow rate across all lanes. Thus, a two-lane (in one direction) multilane highway segment with a 60-mi/h FFS would have an expected capacity of 2 × 2,200 = 4,400 pc/h. This flow would not be uniformly distributed in the two lanes. Thus, one lane could have stable flows in excess of 2,200 pc/h/ln.

LOS FOR MULTILANE HIGHWAY SEGMENTS

Automobile Mode

Automobile LOS is defined by density.

Automobile LOS for multilane highway segments are defined in Exhibit 14-4. Because speeds are constant through a broad range of flow rates, LOS are defined on the basis of density, which is a measure of the proximity of vehicles to each other in the traffic stream.

Exhibit 14-4
Automobile LOS for Multilane Highway Segments

LOS	FFS (mi/h)	Density (pc/mi/ln)
A	All	>0–11
B	All	>11–18
C	All	>18–26
D	All	>26–35
E	60	>35–40
	55	>35–41
	50	>35–43
	45	>35–45
F	Demand Exceeds Capacity	
	60	>40
	55	>41
	50	>43
	45	>45

LOS thresholds for multilane highways are the same as those on freeways for LOS A–D. However, multilane highway capacity (the LOS E–F boundary) occurs at lower densities.

For LOS A through D, the criteria are the same as those for basic freeway segments. This classification is appropriate, since both represent multilane uninterrupted flow. The boundary between LOS E and F, however, represents capacity. For multilane highways, capacity occurs at varying densities, depending on the FFS. The density at capacity ranges from 40 pc/mi/ln for 60-mi/h FFS to 45 pc/mi/ln for 45-mi/h FFS.

LOS F is determined when the demand flow rate exceeds capacity. When this occurs, the methodology does not produce a density estimate. Thus, although density in such cases will be above the thresholds shown, specific values cannot be determined.

Exhibit 14-5 shows LOS thresholds in relation to the base speed–flow curves.

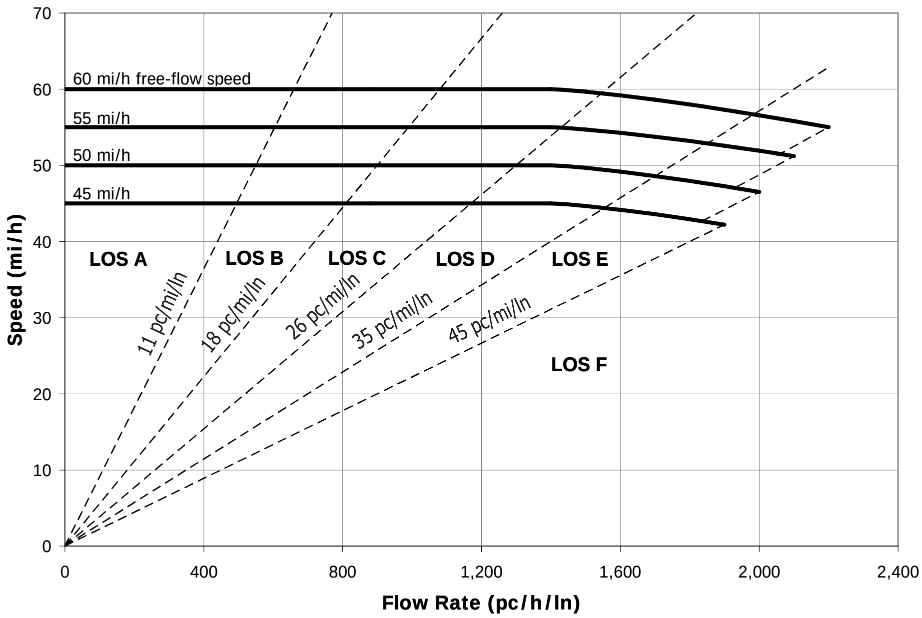


Exhibit 14-5
LOS on Base Speed–Flow Curves

LIVE GRAPH
Click here to view

Automobile LOS Described

The descriptions of LOS for basic freeway segments given in Chapter 11 are also generally applicable to multilane highways. Vehicles entering the highway from a direct access point are an additional factor on multilane highways; these vehicles are not present on basic freeway segments and could result in a breakdown of flow at high flow rates.

The LOS thresholds for multilane highways reflect the collective professional judgment of the members of the Transportation Research Board's Highway Capacity and Quality of Service Committee. The upper values shown for LOS F (40 to 45 pc/mi/ln, depending on the FFS) represent the maximum density at which sustained flows at capacity are expected to occur. Breakdown (LOS F) conditions on multilane highways occur whenever the highway's demand exceeds its capacity.

Bicycle Mode

Bicycle LOS for multilane highway segments are based on a bicycle LOS score, which is in turn based on a traveler-perception index. Chapter 15, Two-Lane Highways, provides details about this index, which is identical for two-lane highways and multilane highways. The LOS ranges for bicycles on multilane highways are given in Exhibit 14-6.

Bicycle LOS is based on a traveler-perception index score. Details are given in Chapter 15.

Exhibit 14-6
Bicycle LOS on Multilane Highways

LOS	Bicycle LOS Score
A	≤1.5
B	>1.5–2.5
C	>2.5–3.5
D	>3.5–4.5
E	>4.5–5.5
F	>5.5

REQUIRED INPUT DATA

Automobile Mode

Analysis of a multilane highway segment requires details concerning the geometric characteristics of the segment and the demand characteristics of the users of the segment. This section presents the required input data for the basic freeway segment methodology; specifics about individual parameters are given in Section 2, Methodology.

Data Describing Multilane Highway Segment

The following information concerning the geometric features of the multilane highway segment is needed to conduct an analysis:

- FFS: 45 to 60 mi/h;
- Number of lanes (one direction): two or three;
- Lane width: 10 ft to more than 12 ft;
- Right-side lateral clearance: 0 ft to more than 6 ft;
- Median- (left-) side lateral clearance: 0 ft to more than 6 ft;
- Access-point density: 0 to 40 points/mi;
- Terrain: level, rolling, or mountainous; or length and percent grade of specific grades; and
- Type of median: divided, TWLTL, or undivided.

Data Describing Demand

The following information is required concerning the users of the multilane highway segment:

- Demand during the analysis hour; or daily demand, *K*-factor, and *D*-factor;
- Heavy-vehicle presence (percent trucks and buses, percent RVs): 0%–100% in general terrain or 0%–25% for specific grades;
- Peak hour factor (PHF): up to 1.00; and
- Driver-population factor: 0.85–1.00.

Length of Analysis Period

The period for any multilane highway analysis is generally the critical 15-min period within the peak hour. The methodology can be applied to any 15-min period, however.

If demand volumes are used, demand flow rates are estimated through the use of the PHF. When 15-min volumes are directly measured, the worst analysis period within the hour is selected, and flow rates are the 15-min volumes multiplied by 4. For subsequent computations in the methodology, the PHF is set to 1.00.

Bicycle Mode

The following data are required to evaluate bicycle LOS on a multilane highway; the ranges of values used in the development of the bicycle LOS model (*2*) are also shown:

- Width of the outside through lane: 10 to 16 ft,
- Shoulder width: 0 to 6 ft,
- Motorized vehicle volumes: up to 36,000 annual average daily traffic (AADT),
- Number of directional through lanes,
- Posted speed: 45 to 50 mi/h,
- Heavy-vehicle percentage: 0% to 2%, and
- Pavement condition: present serviceability rating of 1 to 5.

2. METHODOLOGY

This methodology is used to analyze the capacity, LOS, lane requirements, and impacts of traffic and design features on uninterrupted-flow segments of rural and suburban multilane highways.

LIMITATIONS OF METHODOLOGY

Automobile Mode

The methodology of this chapter does not take into account the following conditions:

- The negative impacts of poor weather conditions, traffic accidents or incidents, railroad crossings, or construction operations;
- Interference caused by parking on the shoulders of the multilane highway;
- The effect of lane drops and lane additions at the beginning or end of multilane highway segments;
- Possible queuing impacts when a multilane highway segment transitions to a two-lane highway segment;
- Differences between various types of median barriers and the difference between the impacts of a median barrier and a TWLTL;
- FFS below 45 mi/h or higher than 60 mi/h;
- Significant presence of on-street parking;
- Presence of bus stops that have significant use; and
- Significant pedestrian activity.

The last three factors are more representative of an urban or suburban arterial, but they may also exist on facilities with more than 2 mi between traffic signals. When the factors are present on uninterrupted-flow segments of multilane highways, the methodology does not deal with their impact on flow. In addition, this methodology cannot be applied to highways with a total of three lanes in both directions, which should be analyzed as two-lane highways with periodic passing lanes.

Uninterrupted-flow facilities that allow access solely through a system of on-ramps and off-ramps from grade separations or service roads should be analyzed as freeways.

Bicycle Mode

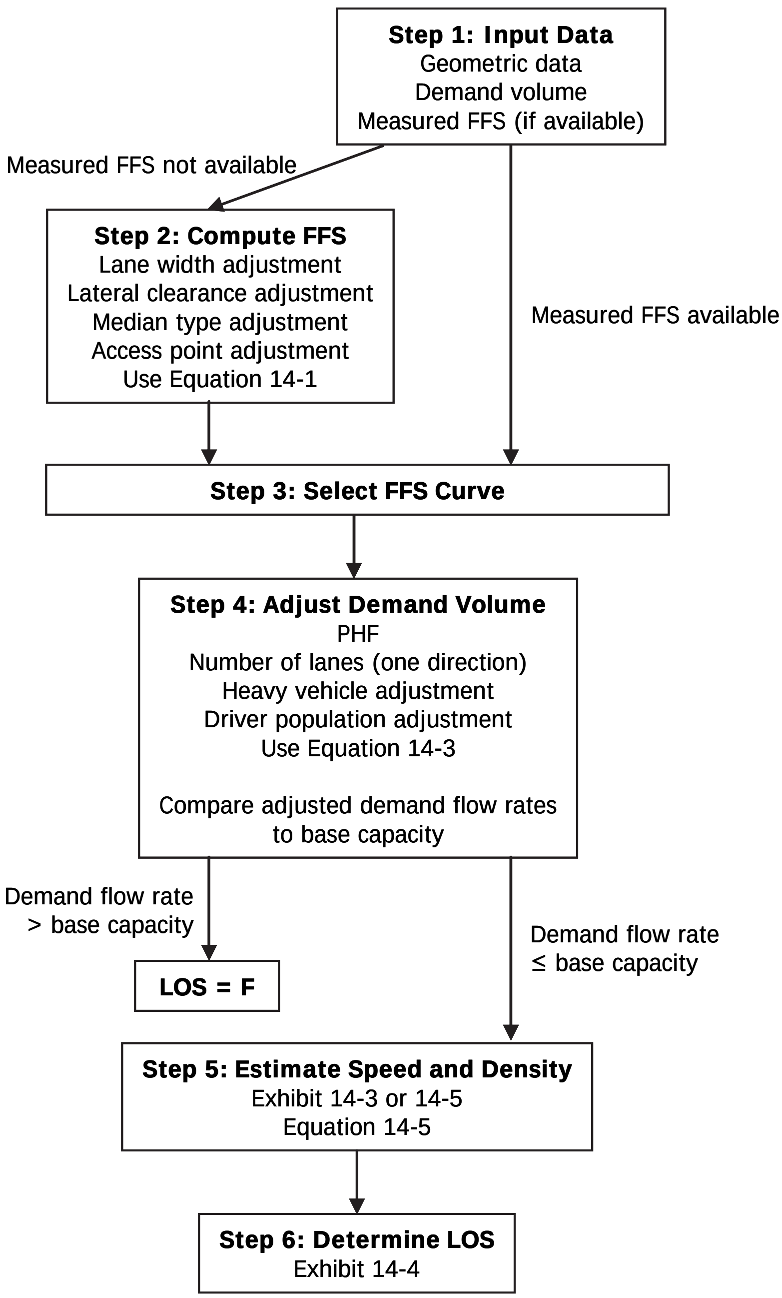
Although the bicycle LOS model has been successfully applied to rural multilane highways, users should be aware that conditions on many of those highways are outside the range of values used to develop the model.

The bicycle methodology was developed with data collected on urban and suburban streets, including facilities that would be defined as suburban multilane highways. Although the methodology has been successfully applied to rural multilane highways in different parts of the United States, users should be aware that conditions on many rural multilane highways (i.e., posted speeds of 55 mi/h or higher or heavy-vehicle percentages over 2%) will be outside the range of values used to develop the bicycle LOS model.

AUTOMOBILE MODE

Exhibit 14-7 provides an overview of this chapter's computational methodology for the automobile mode. It shows a typical operational analysis in which the LOS is determined for a specified set of geometric and traffic conditions. The methodology can also be used, as described in this chapter's Applications section, to determine the number of lanes needed to provide a target LOS, as well as to determine service flow rates, service volumes, and daily service volumes.

Exhibit 14-7
Overview of Multilane Highway Methodology for Automobile Mode



Step 1: Input Data

For a typical operational analysis, the analyst must specify (with either site-specific or default values) demand volume; number and width of lanes; right-side and median lateral clearance; type of median; roadside access points per mile; percent of heavy vehicles, such as trucks, buses, and RVs; PHF; terrain; and driver population factor.

Step 2: Compute FFS

FFS can be determined directly from field measurements or can be estimated as described below.

Field Measurement

FFS is the mean speed of passenger cars during periods of low to moderate flow.

FFS is the mean speed of passenger cars measured during periods of low to moderate flow (up to 1,400 pc/h/ln). For a specific multilane highway segment, speeds are virtually constant in this range of flow rates. If the FFS can be field measured, that determination is preferable. If the FFS is measured directly, no adjustments are applied to the measured value.

The speed study should be conducted at a location representative of the segment at a time when flow rates are less than 1,400 pc/h/ln. The speed study should measure the speeds of all passenger cars or use a systematic sample (e.g., every tenth car in each lane). A sample of at least 100 passenger-car speeds should be obtained. Any speed measurement technique that has been found acceptable for other types of traffic engineering applications may be used. Further guidance on the conduct of speed studies is provided in a standard traffic engineering publication (*3*).

Estimation

It is not possible to make field measurements for future facilities, and field measurement may not be possible or practical for all existing ones. In such cases, the segment's FFS may be estimated by using Equation 14-1, which is based on the physical characteristics of the segment under study:

Equation 14-1

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

where

$BFFS$ = base FFS for multilane highway segment (mi/h);

FFS = FFS of basic freeway segment (mi/h);

f_{LW} = adjustment for lane width, from Exhibit 14-8 (mi/h);

f_{LC} = adjustment for TLC, from Exhibit 14-9 (mi/h);

f_M = adjustment for median type, from Exhibit 14-10 (mi/h); and

f_A = adjustment for access-point density, from Exhibit 14-11 (mi/h).

Base FFS

This methodology covers multilane highway segments with FFS ranging from 45 mi/h to 60 mi/h. The most significant value in Equation 14-1 is the BFFS. There is not a great deal of information available to help establish a base value. In one sense, it is like the design speed—it represents the potential FFS based only on the horizontal and vertical alignment of the highway, not on the impacts of lane widths, lateral clearances, median type, and access points. The design speed may be used as the BFFS if it is available.

Although speed limits are not always uniformly set, the BFFS may be estimated, if necessary, as the posted or statutory speed limit plus 5 mi/h for speed limits 50 mi/h and higher and as the speed limit plus 7 mi/h for speed limits less than 50 mi/h.

Adjustment for Lane Width

Average lane widths less than 12 ft reduce the FFS.

The base condition for lane width is 12 ft or greater. When the average lane width across all lanes is less than 12 ft, the FFS is negatively affected. Adjustments to reflect the effect of narrow average lane widths are shown in Exhibit 14-8.

Exhibit 14-8
Adjustment to FFS for Average Lane Width

Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Adjustment for Lateral Clearance

Clearance restrictions on either the right or left side of the highway reduce the FFS.

The adjustment for lateral clearance on multilane highway segments is based on TLC at the roadside (right side) and at the median (left side). Fixed obstructions with lateral clearance effects include light standards, signs, trees, abutments, bridge rails, traffic barriers, and retaining walls. Standard raised curbs are not considered to be obstructions.

Right-side lateral clearance is measured from the right edge of the travel lanes to the nearest periodic or continuous roadside obstruction. If such obstructions are farther than 6 ft from the edge of the pavement, a value of 6 ft is used.

Left-side lateral clearance is measured from the left edge of the travel lanes to the nearest periodic or continuous obstruction in the median. If such obstructions are farther than 6 ft from the edge of the pavement, a value of 6 ft is used.

Use 6 ft as the left-side clearance for undivided highways and highways with TWLTLs.

Left-side lateral clearances are subject to some judgment. Many types of common median barriers do not affect driver behavior if they are no closer than 2 ft from the edge of the travel lane, including concrete and W-beam barriers. A value of 6 ft would be used in such cases. Also, when the multilane highway segment is undivided or has a TWLTL, no left-side lateral clearance restriction is assumed, and a value of 6 ft is applied because there is a separate adjustment for the type of median that accounts for the impact of an undivided highway on FFS.

Equation 14-2 is used to determine TLC:

Equation 14-2

$$TLC = LC_R + LC_L$$

where

TLC = total lateral clearance (ft) (maximum value 12 ft);

LC_R = right-side lateral clearance (ft) (maximum value 6 ft); and

LC_L = left-side lateral clearance (ft) (maximum value 6 ft).

Exhibit 14-9 shows the reduction in FFS due to lateral obstructions on the multilane highway.

Exhibit 14-9 Adjustment to FFS for Lateral Clearances

Four-Lane Highways		Six-Lane Highways	
TLC (ft)	Reduction in FFS (mi/h)	TLC (ft)	Reduction in FFS (mi/h)
12	0.0	12	0.0
10	0.4	10	0.4
8	0.9	8	0.9
6	1.3	6	1.3
4	1.8	4	1.7
2	3.6	2	2.8
0	5.4	0	3.9

Note: Interpolation to the nearest 0.1 is recommended.

Adjustment for Type of Median

The FFS is reduced on undivided highways.

The adjustment for type of median is given in Exhibit 14-10. Undivided multilane highways reduce the BFFS by 1.6 mi/h.

Exhibit 14-10 Adjustment to FFS for Median Type

Median Type	Reduction in FFS, f_M (mi/h)
Undivided	1.6
TWLTL	0.0
Divided	0.0

Adjustment for Access-Point Density

FFS is reduced as the access-point density increases.

Exhibit 14-11 presents the adjustment to FFS for various levels of access-point density. Studies indicate that for each access point per mile, the estimated FFS decreases by approximately 0.25 mi/h, regardless of the type of median.

The number of access points per mile is determined by dividing the total number of access points (i.e., driveways and unsignalized intersections) on the right side of the highway in the direction of travel by the length of the segment in miles. An intersection or driveway should only be included in the count if it influences traffic flow. Access points that go unnoticed by drivers, or with little activity, should not be used to determine access-point density.

Exhibit 14-11 Adjustment to FFS for Access-Point Density

Access-Point Density (access points/mi)	Reduction in FFS, f_A (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
≥40	10.0

Note: Interpolation to the nearest 0.1 is recommended.

Although the calibration of this adjustment did not include one-way multilane highway segments, it might be appropriate to include intersection approaches and driveways on both sides of the facility in determining the access-point density on one-way segments.

Step 3: Select FFS Curve

As noted previously, once the multilane highway segment's FFS is determined, one of the four base speed–flow curves from Exhibit 14-2 is selected for use in the analysis. Interpolating between curves is not recommended. Criteria for selecting an appropriate curve were given in the text preceding Exhibit 14-2.

Step 4: Adjust Demand Volume

The basic speed–flow curves of Exhibit 14-2 are based on flow rates in equivalent passenger cars per hour, with the driver population dominated by regular users of the multilane highway segment. Demand volumes expressed as vehicles per hour under prevailing conditions must be converted to this basis. Equation 14-3 is used for this adjustment:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

Equation 14-3

where

v_p = demand flow rate under equivalent base conditions (pc/h/ln);

V = demand volume under prevailing conditions (veh/h);

PHF = peak hour factor;

N = number of lanes (one direction);

f_{HV} = adjustment factor for presence of heavy vehicles in traffic stream, from Equation 14-4; and

f_p = adjustment factor for atypical driver populations.

PHF

The PHF represents the variation in traffic flow within an hour. Observations of traffic flow consistently indicate that the flow rates found in the peak 15 min within an hour are not sustained throughout the entire hour. The application of the PHF in Equation 14-3 accounts for this phenomenon.

On multilane highways, typical PHFs range from 0.75 to 0.95. Lower values are typical of lower-volume conditions. Higher values are typical of urban and suburban peak-hour conditions. Field data should be used if possible to develop PHFs that represent local conditions.

Adjustment for Heavy Vehicles

A *heavy vehicle* is defined as any vehicle with more than four wheels on the ground during normal operation. Such vehicles are generally categorized as trucks, buses, or RVs. Trucks cover a wide variety of vehicles, from single-unit trucks with double rear tires to triple-unit tractor–trailer combinations. Small

panel or pickup trucks with only four wheels are, however, classified as passenger cars. Buses include intercity buses, public transit buses, and school buses. Because buses are in many ways similar to single-unit trucks, both types of vehicles are considered in one category. RVs include a wide variety of vehicles from self-contained motor homes to cars and small trucks with trailers (for boats, all-terrain vehicles, or other items). The heavy-vehicle adjustment factor f_{HV} is computed by using Equation 14-4:

Equation 14-4

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of trucks and buses in traffic stream,

P_R = proportion of RVs in traffic stream,

E_T = passenger-car equivalent (PCE) of one truck or bus in traffic stream, and

E_R = PCE of one RV in traffic stream.

The adjustment factor is found in a two-step process. First, the PCE for each truck, bus, and RV is found for the prevailing conditions under study. These equivalency values represent the number of passenger cars that would use the same amount of freeway capacity as one truck, bus, or RV under the prevailing conditions. Second, Equation 14-4 is used to convert the PCE values to the adjustment factor.

In many cases, trucks will be the only heavy vehicle present in the traffic stream. In others, the percentage of RVs will be small compared with trucks and buses. If the ratio of trucks and buses to RVs is 5:1 or greater, all heavy vehicles may be (but do not have to be) considered to be trucks.

The effect of heavy vehicles on traffic flow depends on terrain and grade conditions as well as traffic composition. PCEs can be selected for one of three conditions:

- Extended multilane highway segments in general terrain,
- Specific upgrades, or
- Specific downgrades.

Each of these conditions is more precisely defined and discussed below.

Equivalents for General Terrain Segments

General terrain can be applied where

Grades are ≤2%,

Grades are ≤0.25 mi long, or

Grades are >2% and <3%, and are ≤0.50 mi long.

General terrain refers to extended lengths of multilane highway containing a number of upgrades and downgrades where no single grade is long enough or steep enough to have a significant impact on the operation of the overall segment. As a guideline for this determination, extended-segment analysis can be applied where no one grade of 3% or more is longer than 0.25 mi, or where no single grade between 2% and 3% is longer than 0.50 mi.

There are three categories of general terrain:

- *Level terrain*: Any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars. This type of terrain typically contains short grades of no more than 2%.
- *Rolling terrain*: Any combination of grades and horizontal or vertical alignment that causes heavy vehicles to reduce their speed substantially below that of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length of time or at frequent intervals. *Crawl speed* is the maximum sustained speed that trucks can maintain on an extended upgrade of a given percent. If the grade is long enough, trucks will be forced to decelerate to the crawl speed, which they can maintain for extended distances. Appendix A of Chapter 11, Basic Freeway Segments, contains truck performance curves that provide truck speeds for various lengths and severities of grade. The same curves may be used for uninterrupted-flow segments on multilane highways.
- *Mountainous terrain*: Any combination of grades and horizontal and vertical alignment that causes heavy vehicles to operate at crawl speed for significant distances or at frequent intervals.

Mountainous terrain is relatively rare. Generally, in segments severe enough to cause the type of operation described for mountainous terrain, there will be individual grades that are longer and steeper than the criteria for general terrain analysis.

The mountainous terrain category is rarely used, because individual grades will typically be longer and steeper than the criteria for general terrain analysis.

Exhibit 14-12 shows PCEs for trucks and buses and RVs in general terrain segments.

Exhibit 14-12
PCEs for Heavy Vehicles in General Terrain Segments

Vehicle	PCE by Type of Terrain: Level	Rolling	Mountainous
Trucks and buses, E_T	1.5	2.5	4.5
RVs, E_R	1.2	2.0	4.0

Equivalents for Specific Upgrades

Any grade between 2% and 3% and longer than 0.5 mi, or 3% or greater and longer than 0.25 mi, should be considered to be a separate segment. The analysis of such segments must consider the upgrade conditions and the downgrade conditions separately, as well as whether the grade is a single, isolated grade of constant percentage or part of a series forming a composite grade. Appendix A of Chapter 11 discusses the analysis of composite grades.

Exhibit 14-13 and Exhibit 14-14 give values of E_T and E_R for trucks and buses and for RVs, respectively. These factors vary with the percent of grade, length of grade, and the proportion of heavy vehicles in the traffic stream. Maximum values occur when there are only a few heavy vehicles in the traffic stream. The equivalents decrease as the number of heavy vehicles increases because these vehicles tend to form platoons. Because heavy vehicles have more uniform operating characteristics, fewer large gaps are created in the traffic stream when they platoon, and the impact of a single heavy vehicle in a platoon is less severe than that of a single heavy vehicle in a stream primarily composed of passenger

cars. The aggregate impact of heavy vehicles on the traffic stream, however, increases as the number and percentage of heavy vehicles increase.

Exhibit 14-13
PCEs for Trucks and Buses (E_T) on Upgrades

Percent Upgrade	Length (mi)	Proportion of Trucks and Buses 2%	4%	5%	6%	8%	10%	15%	20%	25%
≤2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
>2 – 3	0.00 – 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.50 – 0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.75 – 1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>1.00 – 1.50	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>1.50	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
>3 – 4	0.00 – 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50 – 0.75	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	>0.75 – 1.00	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	>1.00 – 1.50	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	>1.50	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
>4 – 5	0.00 – 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.50 – 0.75	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	>0.75 – 1.00	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	>1.00	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
>5 – 6	0.00 – 0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.30	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.30 – 0.50	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	>0.50 – 0.75	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	>0.75 – 1.00	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	>1.00	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
>6	0.00 – 0.25	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	1.0
	>0.25 – 0.30	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	>0.30 – 0.50	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	>0.50 – 0.75	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	>0.75 – 1.00	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	>1.00	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

Note: Interpolation for percentage of trucks and buses is recommended to the nearest 0.1.

Exhibit 14-14
PCEs for RVs (E_R) on Upgrades

Percent Upgrade	Length (mi)	Proportion of RVs 2%	4%	5%	6%	8%	10%	15%	20%	25%
≤2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
>2 – 3	0.00 – 0.50	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.50	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
>3 – 4	0.00 – 0.25	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.25 – 0.50	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
>4 – 5	0.00 – 0.25	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 0.50	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
>5	0.00 – 0.25	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	>0.25 – 0.50	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	>0.50	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0

Note: Interpolation for percentage of RVs is recommended to the nearest 0.1.

The grade length should include 25% of the length of the vertical curves at the start and end of the grade.

With two consecutive upgrades, 50% of the length of the vertical curve joining them should be included.

The length of the grade is generally taken from a highway profile. It typically includes the straight portion of the grade plus some portion of the vertical curves at the beginning and end of the grade. It is recommended that 25% of the length of the vertical curves at both ends of the grade be included in the length. Where two consecutive upgrades are present, 50% of the length of the vertical curve joining them is included in the length of each grade.

In the analysis of upgrades, the point of interest is generally at the end of the grade, where heavy vehicles have the maximum effect on operations. However, if a segment ends midgrade (because of a major access point, for example), the length of the grade to the end of the segment would be used.

The point of interest in an analysis of upgrades is usually the spot where heavy vehicles would have the greatest impact on operations: for example, the top of a grade or the top of the steepest grade in a series.

On composite grades, the relative steepness of segments is important. If a 5% upgrade is followed by a 2% upgrade, for example, the maximum impact of heavy vehicles is most likely at the end of the 5% segment. Heavy vehicles would be expected to accelerate after entering the 2% segment.

Equivalents for Specific Downgrades

Knowledge of specific impacts of heavy vehicles on operating conditions on downgrades is limited. In general, if the downgrade is not severe enough to cause trucks to shift into a lower gear (to engage engine braking), heavy vehicles may be treated as if they were on level terrain segments. Where a downgrade is severe, trucks must often use low gears to avoid gaining too much speed and running out of control. In such cases, their effect on operating conditions is more significant than on level terrain. Exhibit 14-15 gives values of E_T for this situation.

Exhibit 14-15
PCEs for Trucks and Buses (E_T) on Specific Downgrades

Percent Downgrade	Length of Grade (mi)	Proportion of Trucks and Buses			
		5%	10%	15%	20%
<4	All	1.5	1.5	1.5	1.5
4 – 5	≤4	1.5	1.5	1.5	1.5
	>4	2.0	2.0	2.0	1.5
>5 – 6	≤4	1.5	1.5	1.5	1.5
	>4	5.5	4.0	4.0	3.0
>6	≤4	1.5	1.5	1.5	1.5
	>4	7.5	6.0	5.5	4.5

On downgrades, RVs are always treated as if they were on level terrain; E_R is therefore always 1.2 on downgrades regardless of the length or severity of the downgrade or the percentage of RVs in the traffic stream.

E_R is always 1.2 on downgrades.

Equivalents for Composite Grades

The vertical alignment of most multilane highways results in a continuous series of grades. It is often necessary to determine the effect of a series of grades in succession. The most straightforward technique is to compute the *average grade,* defined as the total rise from the beginning of the composite grade to the point of interest divided by the length of the grade (to the point of interest).

The average grade technique is an acceptable approach for grades in which all subsections are less than 4% or the total length of the grade is less than 4,000 ft. For more severe composite grades, a detailed technique is presented in Appendix A of Chapter 11, Basic Freeway Segments. This technique uses vehicle performance curves and equivalent speeds to determine the equivalent simple grade for analysis. It can be applied to composite grades on multilane highways.

The average grade can be used when all component grades are <4% or the total length of the grades is <4,000 ft.

Appendix A of Chapter 11 provides a method for addressing more severe composite grades.

Adjustment for Driver Population

The base traffic stream characteristics for multilane highway segments are representative of regular drivers in a traffic stream composed substantially of commuters, or drivers who are familiar with the facility. It is generally accepted

that traffic streams composed of driver populations with different characteristics (e.g., recreational drivers) use freeways less efficiently. Although data are sparse and reported results vary substantially, significantly lower capacities have been reported on weekends, particularly in recreational areas. It may generally be assumed that the reduced capacity (LOS E) extends to service flow rates and service volumes for other LOS as well.

A f_p-value of 1.00 should generally be used, reflective of drivers who are regular users of the freeway.

The adjustment factor f_p is used to reflect the effect of driver population. The values of f_p usually range from 0.85 to 1.00, although lower values have been observed in some cases. In general, the analyst should use a value of 1.00, which reflects commuters or otherwise familiar drivers, unless there is sufficient evidence that a lower value should be used. Where greater accuracy is needed, comparative field studies of commuter and recreational traffic flow and speeds are recommended.

Does LOS F Exist?

At this point, the demand flow rate has been computed and is stated in units of passenger cars per hour per lane under equivalent base conditions. This demand flow rate must be compared with the base capacity (in the same units). If demand exceeds capacity, LOS F is assigned, and the analysis ends. If demand is less than capacity, LOS F does not exist, and the analysis continues.

Step 5: Estimate Speed and Density

At this point in the methodology, the following have been determined: (*a*) the FFS and appropriate FFS curve for use in the analysis, and (*b*) the demand flow rate expressed in passenger cars per hour per lane under equivalent base conditions. With this information, the estimated speed and density of the traffic stream may be determined.

With the equations specified in Exhibit 14-3, the expected mean speed of the traffic stream can be computed. A graphical solution using Exhibit 14-2 can also be performed.

With the estimated speed determined, Equation 14-5 is used to estimate the density of the traffic stream:

Equation 14-5

$$D = \frac{v_p}{S}$$

where

D = density (pc/mi/ln),

v_p = demand flow rate (pc/h/ln), and

S = mean speed of traffic stream (mi/h).

Step 6: Determine LOS

Exhibit 14-4 is entered with the density obtained from Equation 14-5 to determine the expected prevailing LOS.

BICYCLE MODE

The calculation of bicycle LOS on multilane and two-lane highways shares the same methodology, since multilane and two-lane highways operate in fundamentally the same manner for bicyclists and car drivers. Cyclists travel much more slowly than the prevailing traffic flow and stay as far to the right as possible, including using paved shoulders when available. This similarity indicates the need for only one model.

Follow the step-by-step description of the bicycle LOS method given in Chapter 15, Two-Lane Highways, when bicycle LOS on multilane highways is calculated.

The bicycle LOS model for multilane highways uses a traveler perception index calibrated by using a linear regression model. The model fits independent variables associated with roadway characteristics to the results of a user survey that rates the comfort of various bicycle facilities. The resulting bicycle LOS index computes a numerical LOS score, generally ranging from 0.5 to 6.5, which is stratified to produce a LOS A to F result by using Exhibit 14-6.

Full details on the bicycle LOS methodology and calculation procedures are given in Chapter 15, Two-Lane Highways.

3. APPLICATIONS

The analysis methodology for multilane highway segments is relatively straightforward. Thus, it can be directly used in any one of four applications:

1. *Operational analysis*: All traffic and roadway conditions are specified for an existing facility or a future facility with forecast conditions. The existing or expected LOS is determined.
2. *Design analysis*: A forecast demand volume is used, and key design parameters are specified (e.g., lane width and lateral clearance). The number of lanes required to deliver a target LOS is determined.
3. *Planning and preliminary engineering*: The basic scenario is the same as that for design analysis except that the analysis is conducted at a much earlier stage in the development process. Inputs include default values, and the demand volume is usually stated as an AADT.
4. *Service flow rates and service volumes*: The service flow rate, service volume, daily service volume, or all three are estimated for each LOS for an existing or future facility. All traffic and roadway conditions must be specified for this type of analysis.

Because the methodology and its algorithms are simple and do not involve iterations, all of the types of analysis cited can be completed without the iterative approach required by many other HCM methodologies.

DEFAULT VALUES

For this chapter's methodology, a range of input data is needed. Most of these data should be field measured or estimated values for the specific segment under consideration. When some of the data are not available, default values may be used. However, use of default values will affect the accuracy of the output. Exhibit 14-16 shows the data that are required to conduct an operational analysis and the recommended default values when site-specific data are unavailable (*4*).

Exhibit 14-16
Required Input Data and Default Values for Multilane Highway Segments

Required Data	Default Values
Geometric Data	
Number of lanes in one direction	2 or 3 (in one direction), must have site-specific value
Lane width	12 ft
TLC	12 ft
Access-point density	8 access points/mi (rural) 16 access points/mi (low-density suburban) 25 access points/mi (high-density suburban)
Terrain or specific grade (%, length)	No default, must have site-specific value
Base FFS	65 mi/h
Demand Data	
Length of analysis period	15 min
PHF	0.88, rural; 0.95, suburban
Percentage of heavy vehicles	10%, rural; 5%, urban*
Driver population factor	1.00

Note: *Alternative state-specific default values for percentage of heavy vehicles are given in Chapter 26, Freeway and Highway Segments: Supplemental.

The analyst may also replace the default values of Exhibit 14-16 with defaults that have been locally calibrated.

ESTABLISHING ANALYSIS BOUNDARIES

The methodology of this chapter applies to an uninterrupted-flow segment of multilane highway with uniform prevailing conditions. Thus, any point at which one or more of the prevailing conditions change should mark the beginning of a new analysis segment. The following conditions generally necessitate segmenting the highway:

Grade changes of 2% or more, changes in the highway's geometric characteristics, changes in speed limit, signal or STOP control of the highway, and major access points are places where multilane highways should be segmented.

- A change in the basic number of travel lanes on the highway;
- A change in the highway's median treatment;
- A change of grade of 2% or more or a constant upgrade longer than 4,000 ft;
- The presence of a traffic signal, STOP sign, or roundabout along the multilane highway;
- A significant change in the access-point density;
- A change in the speed limit;
- An access point at which a significant number or percentage of vehicles enters or leaves the highway; and
- The presence of a bottleneck condition.

In general, when analysis boundaries are established, the minimum length of a study segment should be 2,500 ft. The boundary of a study segment should be no closer than 0.25 mi to a traffic signal.

TYPES OF ANALYSIS

Operational Analysis

Operational analyses find the expected LOS for specified roadway and traffic conditions.

The operational analysis application was fully specified in the Methodology section of this chapter. Operational analysis begins with all input parameters specified and is used to find the expected LOS that would result from the prevailing roadway and traffic conditions.

Design Analysis

Design analyses find the number of lanes required for a target LOS, given a specified demand volume.

In design analysis, a known demand volume is used to determine the number of lanes needed to deliver a target LOS. Two modifications are required to the operational analysis methodology. First, since the number of lanes is to be determined, the demand volume is converted to a demand flow rate in passenger cars per hour, not passenger cars per hour per lane, by using Equation 14-6 instead of Equation 14-3:

Equation 14-6

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

where v is the demand flow rate in passenger cars per hour and all other variables are as previously defined.

Second, a maximum service flow rate for the target LOS is then selected from Exhibit 14-17. These values are selected from the base speed–flow curves of Exhibit 14-5 for each LOS.

Exhibit 14-17
Maximum Service Flow Rates (pc/h/ln) for Multilane Highway Segments Under Base Conditions

FFS (mi/h)	Target LOS A	B	C	D	E
60	660	1,080	1,550	1,980	2,200
55	600	990	1,430	1,850	2,100
50	550	900	1,300	1,710	2,000
45	290	810	1,170	1,550	1,900

Then the number of lanes required to deliver the target LOS can be found from Equation 14-7:

Equation 14-7

$$N = \frac{v}{MSF_i}$$

where N is the number of lanes required and MSF_i is the maximum service flow rate for LOS i from Exhibit 14-17. Equation 14-6 and Equation 14-7 can be conveniently combined as Equation 14-8:

Equation 14-8

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

where all variables are as previously defined.

All fractional values of N must be rounded up.

The value of N resulting from Equation 14-7 or Equation 14-8 will most likely be fractional. Because only integer numbers of lanes can be constructed, the result is always rounded to the next higher value. Thus, if the result is 2.1 lanes, 3 lanes must be provided. In effect, the minimum number of lanes needed to provide the target LOS is 2.1. If the result were rounded to 2, a poorer LOS than the target value would result.

Because only whole lanes can be built, it may not be possible to achieve the target LOS for a given demand volume.

This rounding-up process will occasionally produce an interesting result: it is possible that a target LOS (for example, LOS C) cannot be achieved for a given demand volume. If 2.1 lanes are required to produce LOS C, providing two lanes would drop the LOS, most likely to D. However, if three lanes are provided, the LOS might actually improve to B. Thus, some judgment may be required to interpret the results. In this case, two lanes might be provided even though they would result in a borderline LOS D. Economic considerations might lead a decision maker to accept a slightly lower operating condition than that originally targeted.

Planning and Preliminary Engineering

Planning and preliminary engineering applications also find the number of lanes required to deliver a target LOS but provide more generalized input values to the methodology.

The objective of planning or preliminary engineering is to get a general idea of the number of lanes that will be required to deliver a target LOS. The primary differences are that many default values will be used, and the demand volume will be usually expressed as an AADT. Thus, a planning and preliminary engineering analysis starts by converting the demand expressed as an AADT to an estimate of the directional peak-hour demand volume (DDHV), as shown in Equation 14-9.

$$V = DDHV = AADT \times K \times D$$

Equation 14-9

Chapter 3, Modal Characteristics, provides additional guidance on K- and D-factors.

where K is the proportion of AADT occurring during the peak hour, D is the proportion of peak-hour volume traveling in the peak direction, and all other variables are as previously defined.

Once the hourly demand volume is estimated, the methodology follows the same path as that for design analysis.

Service Flow Rates, Service Volumes, and Daily Service Volumes

This chapter's methodology can be easily manipulated to produce service flow rates, service volumes, or daily service volumes, or all three, for a multilane highway segment.

Exhibit 14-17 gives values of the maximum service flow rates MSF_i for each LOS for multilane highways of various FFSs. These values are given in terms of passenger cars per hour per lane under equivalent base conditions. A service flow rate SF_i is the maximum rate of flow that can exist while LOS i is maintained during the 15-min analysis period under prevailing conditions. It can be computed from the maximum service flow rate by using Equation 14-10:

$$SF_i = MSF_i \times N \times f_{HV} \times f_p$$

Equation 14-10

where all variables are as previously defined.

A service flow rate can be converted to a service volume SV_i by applying a PHF , as shown in Equation 14-11. A service volume is the maximum hourly volume that can exist while LOS i is maintained during the worst 15-min period of the analysis hour.

$$SV_i = SF_i \times PHF$$

Equation 14-11

where all variables are as previously defined.

A daily service volume DSV_i is the maximum AADT that can be accommodated by the facility under prevailing conditions while LOS i is maintained during the worst 15-min period of the analysis day. It is estimated from Equation 14-12:

$$MSV_i = \frac{SV_i}{K \times D}$$

Equation 14-12

where all variables are as previously defined.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 14-18 and Exhibit 14-19 are generalized daily service volume tables for multilane highway segments or facilities. They are based on a set of specified typical conditions for rural and urban multilane highways:

- Percent HV = 10% (rural), 5% (urban);
- FFS = 60 mi/h;
- PHF = 0.88 (rural), 0.95 (urban); and
- Driver population factor f_p = 1.00.

Values of rural and urban daily service volumes are provided for four-lane and six-lane highways in level and rolling terrain. A range of *K*- and *D*-factors is provided. Users should enter Exhibit 14-18 and Exhibit 14-19 with local or regional values of these factors for the appropriate size of multilane highway in the appropriate terrain.

Exhibit 14-18
Generalized Daily Service Volumes for Rural Multilane Highways (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Highways LOS B	LOS C	LOS D	LOS E	Six-Lane Highways LOS B	LOS C	LOS D	LOS E
Level Terrain									
0.09	0.50	33.2	48.0	63.1	73.8	49.8	71.9	94.6	110.7
	0.55	30.2	43.6	57.4	67.1	45.3	65.4	86.0	100.6
	0.60	27.7	40.0	52.6	61.5	41.5	60.0	78.9	92.2
	0.65	25.5	36.9	48.5	56.8	38.3	55.3	72.8	85.1
0.10	0.50	29.9	43.2	56.8	66.4	44.8	64.8	85.2	99.6
	0.55	27.2	39.2	51.6	60.4	40.8	58.9	77.4	90.6
	0.60	24.9	36.0	47.3	55.3	37.4	54.0	71.0	83.0
	0.65	23.0	33.2	43.7	51.1	34.5	49.8	65.5	76.6
0.11	0.50	27.2	39.2	51.6	60.4	40.8	58.9	77.4	90.6
	0.55	24.7	35.7	46.9	54.9	37.0	53.5	70.4	82.3
	0.60	22.6	32.7	43.0	50.3	34.0	49.1	64.5	75.5
	0.65	20.9	30.2	39.7	46.4	31.3	45.3	59.6	69.7
0.12	0.50	24.9	36.0	47.3	55.3	37.4	54.0	71.0	83.0
	0.55	22.6	32.7	43.0	50.3	34.0	49.1	64.5	75.5
	0.60	20.8	30.0	39.4	46.1	31.1	45.0	59.2	69.2
	0.65	19.2	27.7	36.4	42.6	28.7	41.5	54.6	63.9
Rolling Terrain									
0.09	0.50	29.8	43.1	56.7	66.3	44.7	64.6	85.0	99.4
	0.55	27.1	39.2	51.5	60.3	40.7	58.8	77.3	90.4
	0.60	24.9	35.9	47.2	55.2	37.3	53.9	70.8	82.9
	0.65	22.9	33.1	43.6	51.0	34.4	49.7	65.4	76.5
0.10	0.50	26.8	38.8	51.0	59.7	40.3	58.2	76.5	89.5
	0.55	24.4	35.3	46.4	54.2	36.6	52.9	69.6	81.4
	0.60	22.4	32.3	42.5	49.7	33.6	48.5	63.8	74.6
	0.65	20.7	29.8	39.2	45.9	31.0	44.7	58.9	68.8
0.11	0.50	24.4	35.3	46.4	54.2	36.6	52.9	69.6	81.4
	0.55	22.2	32.0	42.2	49.3	33.3	48.1	63.2	74.0
	0.60	20.3	29.4	38.6	45.2	30.5	44.1	58.0	67.8
	0.65	18.8	27.1	35.7	41.7	28.2	40.7	53.5	62.6
0.12	0.50	22.4	32.3	42.5	49.7	33.6	52.9	63.8	74.6
	0.55	20.3	29.4	38.6	45.2	30.5	48.1	58.0	67.8
	0.60	18.6	26.9	35.4	41.4	28.0	44.1	53.1	62.1
	0.65	17.2	24.9	32.7	38.2	25.8	40.7	49.0	57.4

Note: Key assumptions: 12% trucks, 0.88 PHF, 60-mi/h FFS, driver population factor 1.0.

Exhibit 14-19
Generalized Daily Service Volumes for Urban Multilane Highways (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Highways LOS B	LOS C	LOS D	LOS E	Six-Lane Highways LOS B	LOS C	LOS D	LOS E
Level Terrain									
0.08	0.50	48.3	69.3	88.5	98.4	72.4	104.0	132.8	147.5
	0.55	43.9	63.0	80.5	89.4	65.8	94.5	120.7	134.1
	0.60	40.2	57.8	73.8	82.0	60.4	86.6	110.7	123.0
	0.65	37.1	53.3	68.1	75.7	55.7	80.0	102.1	113.5
0.09	0.50	42.9	61.6	78.7	87.4	64.4	92.4	118.0	131.2
	0.55	39.0	56.0	71.5	79.5	58.5	84.0	107.3	119.2
	0.60	35.8	51.3	65.6	72.9	53.7	77.0	98.4	109.3
	0.65	33.0	47.4	60.5	67.3	49.5	71.1	90.8	100.9
0.10	0.50	38.6	55.4	70.8	78.7	57.9	83.2	106.2	118.0
	0.55	35.1	50.4	64.4	71.5	52.7	75.6	96.6	107.3
	0.60	32.2	46.2	59.0	65.6	48.3	69.3	88.5	98.4
	0.65	29.7	42.6	54.5	60.5	44.6	64.0	81.7	90.8
0.12	0.50	35.1	50.4	64.4	71.5	52.7	75.6	96.6	107.3
	0.55	31.9	45.8	58.5	65.0	47.9	68.7	87.8	97.6
	0.60	29.3	42.0	53.7	59.6	43.9	63.0	80.5	89.4
	0.65	27.0	38.8	49.5	55.0	40.5	58.2	74.3	82.5
Rolling Terrain									
0.08	0.50	44.8	64.4	82.2	91.3	67.3	96.5	123.3	137.0
	0.55	40.8	58.5	74.7	83.0	61.1	87.8	112.1	124.6
	0.60	37.4	53.6	68.5	76.1	56.0	80.4	102.8	114.2
	0.65	34.5	49.5	63.2	70.3	51.7	74.3	94.9	105.4
0.09	0.50	39.9	57.2	73.1	81.2	59.8	85.8	109.6	121.8
	0.55	36.2	52.0	66.4	73.8	54.4	78.0	99.6	110.7
	0.60	33.2	47.7	60.9	67.7	49.8	71.5	91.3	101.5
	0.65	30.7	44.0	56.2	62.5	46.0	66.0	84.3	93.7
0.10	0.50	35.9	51.5	65.8	73.1	53.8	77.2	98.6	109.6
	0.55	32.6	46.8	59.8	66.4	48.9	70.2	89.7	99.6
	0.60	29.9	42.9	54.8	60.9	44.8	64.4	82.2	91.3
	0.65	27.6	39.6	50.6	56.2	41.4	59.4	75.9	84.3
0.12	0.50	32.6	46.8	59.8	66.4	48.9	77.2	89.7	99.6
	0.55	29.6	42.5	54.4	60.4	44.5	70.2	81.5	90.6
	0.60	27.2	39.0	49.8	55.4	40.8	64.4	74.7	83.0
	0.65	25.1	36.0	46.0	51.1	37.6	59.4	69.0	76.6

Note: Key assumptions: 8% trucks, 0.93 PHF, 60-mi/h FFS, driver population factor 1.0.

Exhibit 14-18 and Exhibit 14-19 must be used with care. Because the characteristics of any given multilane highway may or may not be typical, the values should not be used in the analysis of a specific segment of multilane highway. The exhibits are intended to allow a general evaluation of many facilities within a given jurisdiction on a first-pass basis to identify those segments or facilities that might be in need of remediation. Any segments or facilities so identified should then be submitted to specific analysis by using this chapter's methodology and each segment's site-specific characteristics. Exhibit 14-18 and Exhibit 14-19 should not be used to make final decisions on which segments or facilities to upgrade or on the specific designs proposed for such upgrades.

Daily service volumes are computed with Equation 14-10 through Equation 14-12, which combined yield Equation 14-13:

Equation 14-13

$$DSV_i = \frac{MSF_i \times N \times f_{HV} \times f_p \times PHF}{K \times D}$$

where all variables are as previously defined. Values of *MSF* are selected from Exhibit 14-18 or Exhibit 14-19 for the typical FFS of 60 mi/h. Exhibit 14-18 and Exhibit 14-19 do not show LOS A, since this level is rarely of interest in assessing improvement programs.

For multilane highways, daily service volume tables are quite easy to construct by using localized typical values and local defaults. Equation 14-13 is easily applied. All of the variables in the equation simply have to be defined for a given FFS. The heavy-vehicle adjustment depends on PCEs, which are easily obtained for each of the terrain categories.

USE OF ALTERNATIVE TOOLS

Except for the effects of interaction with other facilities, the limitations of the methodology that were stated earlier in the chapter have minimal potential to be addressed by alternative tools. There is thus insufficient experience with alterative tools to support the development of useful guidance for their application to multilane highways.

4. EXAMPLE PROBLEMS

Exhibit 14-20
List of Example Problems

Problem Number	Description	Application
1	LOS on an undivided four-lane highway	Operational analysis
2	LOS on a five-lane highway with TWLTL	Operational analysis
3	Design cross section required to provide target LOS	Design analysis
4	Multilane highway modernization	Planning analysis
5	Future cross section required to provide target LOS	Planning analysis

EXAMPLE PROBLEM 1: LOS ON UNDIVIDED FOUR-LANE HIGHWAY

A 3.25-mi, undivided four-lane highway is primarily on level terrain. The highway does, however, contain one sustained grade of 2.5% that is 3,200 ft long. At what LOS is the highway expected to operate?

The Facts

- Level terrain; 3,200-ft, 2.5% grade included;
- Base FFS = 65 mi/h;
- Lane width: 11 ft;
- Clearance at roadside: 4 ft;
- Access points per mile: 20;
- Peak-hour volume: 1,900 veh/h;
- Traffic composition: 13% trucks, 2% RVs;
- PHF = 0.90; and
- Familiar facility users.

Comments

Three solutions will be needed in this case: (*a*) for the level terrain portion of the highway, (*b*) for the 2.5% upgrade portion of the highway, and (*c*) for the 2.5% downgrade portion of the highway. The factor that will vary for each of these is the heavy-vehicle adjustment factor.

Step 1: Input Data

All input data are specified in the example problem statement.

Step 2: Compute FFS

The FFS is estimated with Equation 14-1. The BFFS is given as 65 mi/h. Adjustments are needed for

- Lane width f_{LW} = 1.9 mi/h (Exhibit 14-8, with 11-ft lanes);
- Lateral clearance f_{LC} = 0.4 mi/h (Exhibit 14-9, with TLC = 4 + 6 = 10 ft, four lanes);
- Median type f_M = 1.6 mi/h (Exhibit 14-10, for undivided highways); and
- Access-point density f_A = 5.0 mi/h (Exhibit 14-11, with 20 access points/mi).

Then

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

$$FFS = 65.0 - 1.9 - 0.4 - 1.6 - 5.0 = 56.1 \text{ mi/h}$$

Step 3: Select FFS Curve

FFSs are all rounded to the nearest 5 mi/h. Therefore, the FFS used in the calculation will be 55 mi/h.

Step 4: Adjust Demand Volume

The demand volume, stated in vehicles per hour under prevailing conditions, must be converted to a demand flow rate in passenger cars per hour under base conditions by using Equation 14-3:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

where

V = demand volume (veh/h) (1,900 veh/h, given),

PHF = peak hour factor (0.90, given), and

f_p = driver population factor (1.00, familiar users).

The heavy-vehicle adjustment factor is computed by using Equation 14-4:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Three sets of PCEs have to be determined: for level terrain, for 2.5% upgrade, and for 2.5% downgrade.

- Level terrain: E_T = 1.5 (Exhibit 14-12); E_R = 1.2 (Exhibit 14-12);
- Upgrade: E_T = 1.5 (Exhibit 14-13, with 13% trucks and a 2.5% grade; 3,200/5,280 = 0.61 mi); E_R = 3.0 (Exhibit 14-14 with 2% RVs and 2.5% grade for 0.61 mi); and
- Downgrade: E_T = 1.5 (Exhibit 14-15 with less than 4% grade and 13% trucks); E_R = 1.2 (from the text following Exhibit 14-15).

In this case, the equivalents are the same for the level terrain segments and the 2.5% downgrade. Consequently, there are only two different heavy-vehicle adjustment factors to work with:

$$f_{HV}(\text{level, down}) = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(1.2 - 1)} = 0.935$$

$$f_{HV}(\text{up}) = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(3.0 - 1)} = 0.905$$

There are thus two values of flow rate in passenger cars per hour under base conditions:

$$v_p\,(\text{level, down}) = \frac{1{,}900}{0.90 \times 2 \times 0.935 \times 1} = 1{,}129 \text{ pc/h}$$

$$v_p\,(\text{up}) = \frac{1{,}900}{0.90 \times 2 \times 0.905 \times 1} = 1{,}166 \text{ pc/h}$$

Step 5: Estimate Speed and Density

Speed for the two demand levels can be estimated by using the equations in Exhibit 14-3 or graphically by using Exhibit 14-5. In this case, both demand flow rates, 1,129 pc/h and 1,166 pc/h, are less than 1,400 pc/h. From Exhibit 14-3, the speed for both of these situations is the FFS, or 55 mi/h.

The densities of the general-terrain and specific-grade segments are then computed with Equation 14-5:

$$D = \frac{v_p}{S}$$

$$D\,(\text{level, down}) = \frac{1{,}129}{55} = 20.5 \text{ pc/mi/ln}$$

$$D\,(\text{up}) = \frac{1{,}166}{55} = 21.2 \text{ pc/mi/ln}$$

Step 6: Determine LOS

As shown in Exhibit 14-4, the LOS for both densities is C.

Discussion

The multilane highway described here operates at LOS C throughout the entire study area, including the upgrade and the downgrade within it. In a sense, this problem involves a *facility* as opposed to a *segment*. The facility contains several component segments: level-terrain segments on either side of the 2.5%, 3,200-ft grade and the uphill and downhill portions of the 2.5% grade itself.

This chapter's methodology applies to uniform multilane highway segments. In this example problem, there were three segments, which together formed a facility of more than 3 mi. LOS C on all segments is very likely acceptable and would not generally call for immediate remediation.

EXAMPLE PROBLEM 2: LOS ON FIVE-LANE HIGHWAY WITH TWLTL

An 11,000-ft segment of a five-lane highway (two travel lanes in each direction plus a TWLTL) includes a 4% grade of 6,000 ft followed by 5,000 ft of level terrain. At what LOS is the facility expected to operate?

The Facts

- Lane width: 12 ft;
- Lateral clearance, both sides of the roadway: 12 ft;

- Traffic composition: 6% trucks, 0% RVs;
- Access points per mile on the level segment: eastbound, 10; westbound, 13;
- Access points per mile on the specific grade segment: eastbound, 10; westbound, 0;
- PHF = 0.90;
- Familiar users of the facility;
- Peak-hour demand: 1,500 veh/h;
- The upgrade occurs in the westbound direction; and
- Posted speed limit = 45 mi/h.

Comments

This problem is similar to Example Problem 1 in that there are three segments in the facility as described, each of which must be analyzed. The upgrade and downgrade on the 4% grade must be separately analyzed, as well as the level terrain segment. This case is a bit more complex, since not all characteristics of the segments are the same, particularly access points. Because no BFFS is given, it will be estimated as the speed limit plus 7 mi/h, or 45 + 7 = 52 mi/h.

Step 1: Input Data

All input data are given in the example problem statement.

Step 2: Compute FFS

The FFS is estimated by using Equation 14-1:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

In this case, the BFFS is estimated to be 52 mi/h. The lane width is 12 ft, which is the base condition; therefore f_{LW} = 0.0 mi/h (Exhibit 14-8). The lateral clearance is 12 ft at each roadside, but a maximum value of 6 ft may be used. A TWLTL is considered to have a median lateral clearance of 6 ft. Thus, the TLC is 6 + 6 = 12 ft, which is also a base condition. Therefore, f_{LC} = 0.0 mi/h (Exhibit 14-9). The median type adjustment f_M is also 0.0 mi/h (Exhibit 14-10).

For this example problem, only the access-point density produces a nonzero adjustment to the BFFS. Both eastbound (EB) segments (level terrain, 4% downgrade) have 10 access points/mi. From Exhibit 14-11, the corresponding adjustment factor is 2.5 mi/h. The westbound (WB) level-terrain segment has 13 access points/mi, and an adjustment factor of 3.3 mi/h (by interpolation in Exhibit 14-11). The WB upgrade has 0 access points/mi and an adjustment factor of 0.0 mi/h. Therefore

$$FFS_{\text{EB,Level,Downgrade}} = 52.0 - 0.0 - 0.0 - 0.0 - 2.5 = 49.5 \text{ mi/h}$$

$$FFS_{\text{WB,Level}} = 52.0 - 0.0 - 0.0 - 0.0 - 3.3 = 48.7 \text{ mi/h}$$

$$FFS_{\text{WB,Upgrade}} = 52.0 - 0.0 - 0.0 - 0.0 - 0.0 = 52.0 \text{ mi/h}$$

Step 3: Select FFS Curve

Despite the fact that slight differences in FFS exist, all three segment analyses will use the 50-mi/h speed–flow curve.

Step 4: Adjust Demand Volume

Demand volume is adjusted by using Equation 14-3:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

where

V = 1,500 veh/h (given),

PHF = 0.90 (given),

N = 2 lanes (given), and

f_p = 1.00 (familiar users, given).

To compute the heavy-vehicle adjustment factor (Equation 14-4), PCEs for trucks are needed for (*a*) level terrain, (*b*) a 4%, 6,000-ft upgrade, and (*c*) a 4%, 6,000-ft downgrade. The following values are obtained:

- Level terrain: 1.5 (Exhibit 14-12);
- Upgrade: 3.0 (Exhibit 14-13, with 6% trucks and a 4% grade, 6,000/5,280 = 1.14 mi); and
- Downgrade: 1.5 (Exhibit 14-15 with a 4% grade less than 4 mi long).

As in Example Problem 1, the downgrade equivalent is the same as the equivalent for level terrain. Therefore, there are only two heavy-vehicle adjustment factors (Equation 14-4):

$$f_{HV}\,(\text{level,down}) = \frac{1}{1+0.06(1.5-1)} = 0.971$$

$$f_{HV}\,(\text{up}) = \frac{1}{1+0.06(3.0-1)} = 0.893$$

and

$$v_p\,(\text{level, down}) = \frac{1500}{0.90 \times 2 \times 0.971 \times 1.0} = 858 \text{ pc/mi/ln}$$

$$v_p\,(\text{up}) = \frac{1500}{0.90 \times 2 \times 0.893 \times 1.0} = 933 \text{ pc/mi/ln}$$

Step 5: Estimate Speed and Density

Speed is estimated by using the equations of Exhibit 14-3 or the graph in Exhibit 14-5. With the equations of Exhibit 14-3, both demand flow rates are less than 1,400 pc/h/ln. Therefore, the speeds are equal to the FFSs, both of which are 50 mi/h.

Density is computed by using Equation 14-5:

$$D\ (\text{level, down}) = \frac{858}{50} = 17.1\ \text{pc/mi/ln}$$

$$D\ (\text{up}) = \frac{933}{50} = 18.7\ \text{pc/mi/ln}$$

Step 6: Determine LOS

The LOS is found by comparing the densities of the segments with the criteria in Exhibit 14-4. The level terrain and downgrade segments operate at LOS B. The upgrade segment operates at LOS C.

Discussion

Even though the upgrade technically operates at LOS C, it is very close to the LOS B boundary (18.0 pc/mi/ln). All segments of the multilane highway facility described operate well. No remediation would likely be needed.

EXAMPLE PROBLEM 3: DESIGN CROSS SECTION REQUIRED TO PROVIDE TARGET LOS

A new 2-mi segment of multilane highway will be built within a 150-ft right-of-way. Sixty feet of right-of-way will be reserved for clear zones; therefore, 90 ft of width will be available for travel lanes, shoulders, and median. How many travel lanes are needed to provide LOS D during the peak hour?

The Facts

- AADT = 60,000 veh/day;
- $D = 0.10$; $K = 0.55$;
- 50-mi/h speed limit;
- Rolling terrain;
- Traffic composition: 5% trucks, no RVs;
- PHF = 0.90;
- Access-point density = 10.0 access points/mi; and
- Familiar facility users.

Comments

This problem is potentially iterative. The exact cross section is unknown—not only the number of lanes but also the lateral clearances and median treatment. Thus, assumptions will be made and will have to be checked when the trial analysis is complete. To begin the solution, it will be assumed that 12-ft lanes will be provided and that 6-ft clearances at the roadside and median will also be provided (TLC = 6 + 6 = 12 ft). A divided highway will also be assumed for initial computations.

Step 1: Input Data

All input data are specified in the example problem statement.

Step 2: Compute FFS

No BFFS is given. It will be assumed that the BFFS will be 5 mi/h more than the posted speed limit, or 50 + 5 = 55 mi/h. FFS is estimated by using Equation 14-1. Given that the lane width, lateral clearance, and median treatments assumed are all base conditions, there is no adjustment for these. The only adjustment is for access-point density. On the basis of Exhibit 14-11 for 10 access points/mi, the adjustment is 2.5 mi/h. Therefore

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

$$FFS = 55.0 - 0.0 - 0.0 - 0.0 - 2.5 = 52.5 \text{ mi/h}$$

Step 3: Select FFS Curve

Following this chapter's guidelines, for a FFS of 52.5 mi/h, the 55-mi/h base speed–flow curve will be used in this analysis.

Step 4: Determine Number of Lanes Needed for LOS D

Step 4 has a number of intermediate computations. First, the demand volume is stated as an AADT. This volume must be converted to an estimated directional design-hour volume ($V = DDHV$) by using Equation 14-9:

$$V = DDHV = AADT \times K \times D$$

$$V = DDHV = 60{,}000 \times 0.10 \times 0.55 = 3{,}300 \text{ veh/h}$$

The number of lanes required to meet a target LOS is estimated with Equation 14-8:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

where

MSF = 1,850 pc/h/ln (Exhibit 14-17, with LOS D and 55 mi/h);

PHF = 0.90 (given); and

f_p = 1.00 (familiar users).

The heavy-vehicle adjustment factor is estimated by using Equation 14-4 and the PCE for trucks in rolling terrain from Exhibit 14-12, which is 2.5. Then

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1)} = 0.930$$

and

$$N = \frac{3{,}300}{1{,}850 \times 0.90 \times 0.93 \times 1.00} = 2.13 \text{ lanes}$$

This result means that to meet the criteria for LOS D within the peak hour, three lanes in each direction will have to be provided and the facility will be a six-lane multilane highway.

It is necessary to consider whether the assumed cross section (now known to be six lanes) can fit in the 90-ft available width. Lane widths are 12 ft, with 6-ft

lateral clearances at both roadsides and in the median. If a 12-ft median is assumed, the total width becomes (6 × 12) + (6 × 2) + 12 = 72 + 12 + 12 = 96 ft, which is in excess of the 90-ft right-of-way.

The median treatment could be reconsidered. It is not necessary to have a 12-ft median with 6-ft clearances to the inside edges of the travel pavement to produce the desired operation. A concrete median barrier with 2-ft buffers on each side would occupy a total of only 6 ft and would not be expected to have any impact on driver behavior or the FFS. Then the total width required would be (6 × 12) + (6 × 2) + 6 = 90 ft. This is the design cross section. None of the calculations done to this point would be altered by this design.

It is likely that providing a six-lane highway will result in better operations than the minimums of LOS D. With the number of lanes known, the demand flow rate under base conditions can be computed with Equation 14-3:

$$v_p = \frac{3{,}300}{0.90 \times 3 \times 0.93 \times 1.00} = 1{,}314 \text{ pc/h/ln}$$

Step 5: Estimate Speed and Density

The speed of the traffic stream can be determined by using the equations of Exhibit 14-3 or the graph in Exhibit 14-5. On the basis of the equations, because the demand flow rate is less than 1,400 pc/h/ln, the speed is the FFS, or 55 mi/h in this case.

The density may now be computed by using Equation 14-5:

$$D = \frac{1{,}314}{55} = 23.9 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From the criteria of Exhibit 14-4, the LOS provided is C, one grade better than the design target of LOS D.

Discussion

The design resulted in a six-lane cross section on a divided multilane highway with no clearance obstructions. The LOS provided is better than the design target, since three lanes were provided in each direction while only 2.13 lanes were necessary.

EXAMPLE PROBLEM 4: MULTILANE HIGHWAY MODERNIZATION

A 2.5-mi segment of a substandard multilane highway is to be improved by providing wider shoulders, widening the lanes to 12 ft, improving the alignment on a few sharp curves, restricting the number of roadside access points, and adding a median. These improvements will increase the FFS of the facility from 50 mi/h to 60 mi/h. How much additional traffic can be accommodated while the postimprovement LOS is maintained?

The Facts

- Demand flow rate = 1,400 pc/h/ln under base conditions.

Comments

This problem is relatively straightforward. Most of the steps in a standard analysis can be skipped, since the present and future FFS are given, and the demand flow rate has already been reduced to base conditions.

Step 1: Find Existing LOS

With the equations of Exhibit 14-3, the speed of vehicles in the existing configuration will be the FFS, or 50 mi/h. With Equation 14-5, the density is computed as

$$D = \frac{1{,}400}{50} = 28 \text{ pc/mi/ln}$$

From Exhibit 14-4, this is LOS D.

Step 2: Find Expected LOS After Improvement

With the equations of Exhibit 14-3, the speed of vehicles on the improved cross section will also be the FFS, or 60 mi/h. The density is computed with Equation 14-5:

$$D = \frac{1{,}400}{60} = 23.2 \text{ pc/mi/ln}$$

From Exhibit 14-4, this is LOS C.

Step 3: Find Additional Volume Under LOS C

From Exhibit 14-5, the maximum service flow rate for LOS C on a 60-mi/h multilane highway is 1,550 pc/h/ln. The existing demand flow rate is 1,400 pc/h/ln. The additional demand flow possible while LOS C is maintained is 1,550 – 1,400 = 150 pc/h/ln. Since there are three lanes in each direction, the demand flow rate can increase by 3 × 150 = 450 pc/h without slipping into LOS D.

Discussion

This example problem illustrates how the methodology can be adapted to different uses, in this case, evaluating the impact of a proposed improvement to a multilane highway. The LOS improves from D to C, and an additional peak-hour demand flow rate of 450 pc/h can be accommodated by the improved highway while LOS C is maintained.

EXAMPLE PROBLEM 5: FUTURE CROSS SECTION REQUIRED TO PROVIDE TARGET LOS

A new suburban multilane highway is being planned. The opening-day forecast AADT is 42,000 vehicles per day. How many lanes will be needed to provide for LOS C during the peak hour on opening day?

The Facts

Since this is a planning application, many details cannot be based on current information. As a result, some of the “facts” are forecasts, and default values based on regional data are used to complete the list of facts needed for the analysis.

- Demand = 42,000 veh/day;
- $K = 0.10$; $D = 0.60$;
- Traffic composition: 10% trucks, no RVs;
- Rolling terrain;
- Base FFS = 55 mi/h;
- Lane width: 12 ft; roadside lateral clearance: 6 ft;
- Undivided highway;
- Access-point density = 6 access points/mi;
- PHF = 0.90; and
- Commuter traffic.

Comments

The demand volume, given as an AADT, must be converted to a DDHV. Once this is done, the example becomes a design application to determine the number of lanes needed to deliver LOS C.

Step 1: Input Data

All input data for this problem are specified in the problem statement.

Step 2: Compute FFS

The FFS is computed by using Equation 14-1. The BFFS is given. Lane widths and lateral clearances conform to base conditions, and no adjustments will be necessary. Then

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

where

$BFFS$ = 55 mi/h (given);

f_{LW} = 0.0 mi/h (Exhibit 14-8, with 12-ft lanes);

f_{LC} = 0.0 mi/h (Exhibit 14-9, with 6-ft clearances);

f_M = 1.6 mi/h (Exhibit 14-10, undivided);

f_A = 1.5 mi/h (Exhibit 14-11, with 6 access points/mi, interpolated);

and

$$FFS = 55.0 - 0.0 - 0.0 - 1.6 - 1.5 = 51.9 \text{ mi/h}$$

Step 3: Select FFS Curve

On the basis of criteria given in the methodology section, the 50-mi/h FFS curve will be used for this solution.

Step 4: Determine Number of Lanes Needed to Provide LOS C

The number of lanes needed to deliver LOS C on opening day is estimated with Equation 14-8:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

The demand volume must be converted to an hourly basis by using Equation 14-9:

$$V = DDHV = AADT \times K \times D$$
$$V = DDHV = 42{,}000 \times 0.10 \times 0.60 = 2{,}520 \text{ veh/h}$$

The value of MSF is selected from Exhibit 14-17 for a 50-mi/h highway and LOS C: 1,300 pc/h. The heavy-vehicle adjustment factor f_{HV} is computed by using Equation 14-4 with a PCE of 2.5 selected from Exhibit 14-12 for trucks in rolling terrain:

$$f_{HV} = \frac{1}{1 + 0.10(2.5 - 1)} = 0.870$$

Because the demand volume is composed primarily of commuters, the adjustment factor for driver population f_p is 1.00. The PHF was given as 0.90. Then

$$N = \frac{2{,}520}{1{,}300 \times 0.90 \times 0.87 \times 1.00} = 2.48 \text{ lanes}$$

This result implies that a six-lane cross section will have to be provided. Because this cross section is more than the minimum computed, the actual demand flow rate under base conditions should be computed by using Equation 14-3:

$$v_p = \frac{2{,}520}{0.90 \times 3 \times 0.87 \times 1.00} = 1{,}073 \text{ pc/mi/ln}$$

Step 5: Estimate Speed and Density

From the equations of Exhibit 14-3, the expected speed for the demand flow rate is the FFS of 50 mi/h. The density can now be computed with Equation 14-5:

$$D = \frac{1{,}073}{50} = 21.5 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 14-4, the six-lane multilane highway will be expected to operate at LOS C, which was the design objective.

Discussion

In this case, the target LOS has been achieved with the six-lane cross section. The highway will, however, operate in the better portion of LOS C instead of at the boundary.

5. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

1. Reilly, W., D. Harwood, J. Schoen, and M. Holling. *Capacity and LOS Procedures for Rural and Urban Multilane Highways*. NCHRP Project 3-33, Final Report. JHK & Associates, Tucson, Ariz., May 1990.
2. Landis, B. W., V. R. Vattikuti, and M. T. Brannick. Real-Time Human Perceptions: Toward a Bicycle Level of Service. In *Transportation Research Record 1578,* Transportation Research Board, National Research Council, Washington, D.C., 1997, pp. 119–126.
3. Robertson, H. D. (ed.). *Manual of Traffic Engineering Studies*. Institute of Transportation Engineers, Washington, D.C., 2000.
4. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses*. Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 15
TWO-LANE HIGHWAYS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Two-lane highways have one lane for the use of traffic in each direction. The principal characteristic that separates motor vehicle traffic on two-lane highways from other uninterrupted-flow facilities is that passing maneuvers take place in the opposing lane of traffic. Passing maneuvers are limited by the availability of gaps in the opposing traffic stream and by the availability of sufficient sight distance for a driver to discern the approach of an opposing vehicle safely. As demand flows and geometric restrictions increase, opportunities to pass decrease. This creates platoons within the traffic stream, with trailing vehicles subject to additional delay because of the inability to pass the lead vehicles.

Two-lane highways have one lane for the use of traffic in each direction. Passing takes place in the opposing lane of traffic when sight distance is appropriate and safe gaps in the opposing traffic stream are available.

Because passing capacity decreases as passing demand increases, two-lane highways exhibit a unique characteristic: operating quality often decreases precipitously as demand flow increases, and operations can become "unacceptable" at relatively low volume-to-capacity ratios. For this reason, few two-lane highways ever operate at flow rates approaching capacity; in most cases, poor operating quality has led to improvements or reconstruction long before capacity demand is reached.

The quality of service for bicycles is primarily affected by the speed and volume of adjacent traffic flows and by the degree of separation between bicyclist and motor vehicle traffic allowed by the roadway geometry.

Chapter 15, Two-Lane Highways, presents methodologies for the analysis, design, and planning of two-lane highway facilities operating under uninterrupted flow, for both automobiles and bicycles. Uninterrupted flow exists when there are no traffic control devices that interrupt traffic and where no platoons are formed by upstream signals. In general, any segment that is 2.0 to 3.0 mi from the nearest signalized intersection would fit into this category. Where signalized intersections are less than 2.0 mi apart, the facility should be classified as an urban street and analyzed with the methodologies of Chapter 16, Urban Street Facilities, and Chapter 17, Urban Street Segments, which are located in Volume 3. It is assumed that no passing in the opposing lane occurs on urban streets.

Chapter 15 also includes a methodology for predicting the effect of passing and truck climbing lanes on two-lane highways.

CHARACTERISTICS OF TWO-LANE HIGHWAYS

Functions of Two-Lane Highways in Highway Systems

Two-lane highways are a key element in the highway systems of most states and counties. They are located in many different geographical areas and serve a wide variety of traffic functions. Two-lane highways also serve a number of bicycle trips, particularly recreational trips. Any consideration of operating quality criteria must account for these disparate functions.

The functions of two-lane highways include efficient mobility, accessibility, scenic and recreational enjoyment, and service to small towns and communities.

Efficient mobility is the principal function of major two-lane highways that connect major trip generators or that serve as primary links in state and national highway networks. These routes tend to serve long-distance commercial and recreational travelers, and long sections may pass through rural areas without traffic control interruptions. Consistent high-speed operations and infrequent passing delays are desirable for these types of facilities.

Other paved, two-lane rural highways primarily provide *accessibility* to remote or sparsely populated areas. Such highways provide reliable all-weather access and often serve low traffic demands. Cost-effective access is a primary concern. Although high speed is beneficial, it is not the principal objective. Delay, as indicated by the formation of platoons, is a more relevant measure of service quality.

Two-lane roads also serve *scenic and recreational* areas in which the vista and environment are meant to be experienced and enjoyed without traffic interruption or delay. High-speed operation is neither expected nor desired. Passing delays, however, significantly distract from the scenic enjoyment of trips and should be minimized whenever possible.

Two-lane roads may also pass through and serve *small towns and communities*. Such areas have higher-density development than would normally be expected along a rural highway, and speed limits in these areas are often lower. In these cases, drivers expect to be able to maintain speeds close to the posted limit. Since two-lane highway segments serving such developed areas are usually of limited length, passing delays are not a significant issue.

Two-lane highways serve a wide range of functions and serve a variety of rural areas, as well as more developed areas. Therefore, this chapter's methodology and level of service (LOS) criteria provide flexibility to encompass the resulting range of driver expectations.

Classification of Two-Lane Highways

Because of the wide range of functions served by two-lane highways, the automobile methodology establishes three classes of highways.

The first two classes address *rural two-lane highways*. The methodology for them was developed as part of National Cooperative Highway Research Program (NCHRP) Project 3-55(3) in 1999 (*1*) and revised as part of NCHRP Project 20-7(160) in 2003 (*2*).

The third class addresses two-lane highways in *developed areas*. The analysis approach for these highways is a modification of the rural highway method noted previously and was developed by the Florida Department of Transportation (FDOT) (*3*). This modification has not been subjected to a national calibration study and is based on the procedure developed and adopted by FDOT. It is presented here as an alternative procedure, since it is based entirely on information collected in Florida. For clarity, however, the material is integrated into the overall presentation and is not discussed separately as an alternative procedure.

The three classes of two-lane highways are defined as follows:

- *Class I two-lane highways* are highways where motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks are generally assigned to Class I. These facilities serve mostly long-distance trips or provide the connections between facilities that serve long-distance trips.
- *Class II two-lane highways* are highways where motorists do not necessarily expect to travel at high speeds. Two-lane highways functioning as access routes to Class I facilities, serving as scenic or recreational routes (and not as primary arterials), or passing through rugged terrain (where high-speed operation would be impossible) are assigned to Class II. Class II facilities most often serve relatively short trips, the beginning or ending portions of longer trips, or trips for which sightseeing plays a significant role.
- *Class III two-lane highways* are highways serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns or developed recreational areas. On such segments, local traffic often mixes with through traffic, and the density of unsignalized roadside access points is noticeably higher than in a purely rural area. Class III highways may also be longer segments passing through more spread-out recreational areas, also with increased roadside densities. Such segments are often accompanied by reduced speed limits that reflect the higher activity level.

Exhibit 15-1 shows examples of the three classes of two-lane highway.

The definition of two-lane highway classes is based on their function. Most arterials or trunk roads are considered to be Class I highways, while most collectors and local roads are considered to be Class II or Class III highways. The primary determinant of a facility's classification is the motorist's expectation, which might not agree with the overall functional category of the route. For example, a major intercity route passing through a rugged mountainous area might be described as Class II if drivers recognize that high-speed operation is not feasible due to the terrain, but the route could still be considered to be in Class I.

Even Class III highways incorporate only uninterrupted-flow segments of two-lane highways. Occasional signalized or unsignalized intersections on any two-lane highway must be separately analyzed with the appropriate *Highway Capacity Manual* (HCM) methodologies in Chapter 18, Signalized Intersections, Chapter 20, All-Way STOP-Controlled Intersections, or Chapter 21, Roundabouts. The results must be carefully considered in conjunction with those of uninterrupted-flow portions of the facility to obtain a complete picture of probable operations.

Exhibit 15-1
Two-Lane Highway Classification Illustrated

(a) Examples of Class I Two-Lane Highways

(b) Examples of Class II Two-Lane Highways

(c) Examples of Class III Two-Lane Highways

Base Conditions

The base conditions for two-lane highways are the absence of restrictive geometric, traffic, or environmental factors. Base conditions are not the same as typical or default conditions, both of which may reflect common restrictions. Base conditions are closer to what may be considered as ideal conditions (i.e., the best conditions that can be expected given normal design and operational practice). The methodology of this chapter accounts for the effects of geometric, traffic, and environmental factors that are more restrictive than the base conditions. The base conditions for two-lane highways are as follows:

- Lane widths greater than or equal to 12 ft,
- Clear shoulders wider than or equal to 6 ft,
- No no-passing zones,
- All passenger cars in the traffic stream,

- Level terrain, and
- No impediments to through traffic (e.g., traffic signals, turning vehicles).

Traffic can operate ideally only if lanes and shoulders are wide enough not to constrain speeds. Lanes narrower than 12 ft and shoulders narrower than 6 ft have been shown to reduce speeds, and they may increase percent time-spent-following (PTSF) as well.

The length and frequency of no-passing zones are a result of the roadway alignment. No-passing zones may be marked by barrier centerlines in one or both directions, but any segment with a passing sight distance of less than 1,000 ft should also be considered to be a no-passing zone.

On a two-lane highway, passing in the opposing lane of flow may be necessary. It is the only way to fill gaps forming in front of slow-moving vehicles in the traffic stream. Restrictions on the ability to pass significantly increase the rate at which platoons form in the traffic stream, since motorists are unable to pass slower vehicles in front of them.

Basic Relationships

Exhibit 15-2 shows the relationships among flow rate, average travel speed (ATS), and PTSF for an extended directional segment of two-lane highway under base conditions. While the two directions of flow interact on a two-lane highway (because of passing maneuvers), the methodology of this chapter analyzes each direction separately.

Exhibit 15-2(b) illustrates a critical characteristic that affects two-lane highways. Low directional volumes create high values of PTSF. With only 800 pc/h, PTSF ranges from 60% (with 200 pc/h opposing flow) to almost 80% (with 1,600 pc/h opposing flow).

In multilane uninterrupted flow, typically acceptable speeds can be maintained at relatively high proportions of capacity. On two-lane highways, service quality (as measured by PTSF) begins to deteriorate at relatively low demand flows.

CAPACITY AND LOS

Capacity

The capacity of a two-lane highway under base conditions is 1,700 pc/h in one direction, with a limit of 3,200 pc/h for the total of the two directions. Because of the interactions between directional flows, when a capacity of 1,700 pc/h is reached in one direction, the maximum opposing flow would be limited to 1,500 pc/h.

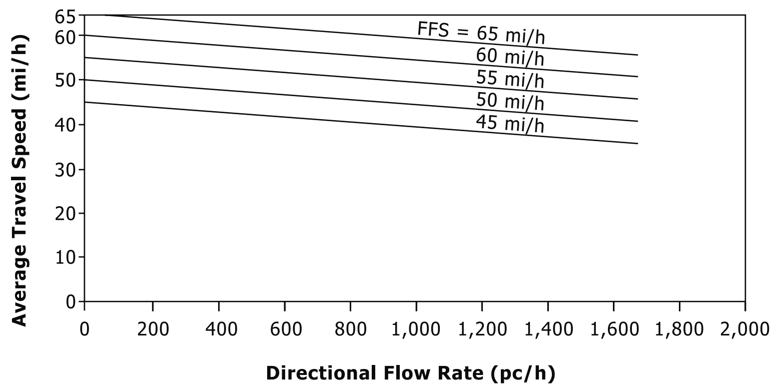
Capacity of a two-lane highway under base conditions is 1,700 pc/h in one direction, with a maximum of 3,200 pc/h in the two directions.

Capacity conditions, however, are rarely observed—except in short segments. Because service quality deteriorates at relatively low demand flow rates, most two-lane highways are upgraded before demand approaches capacity.

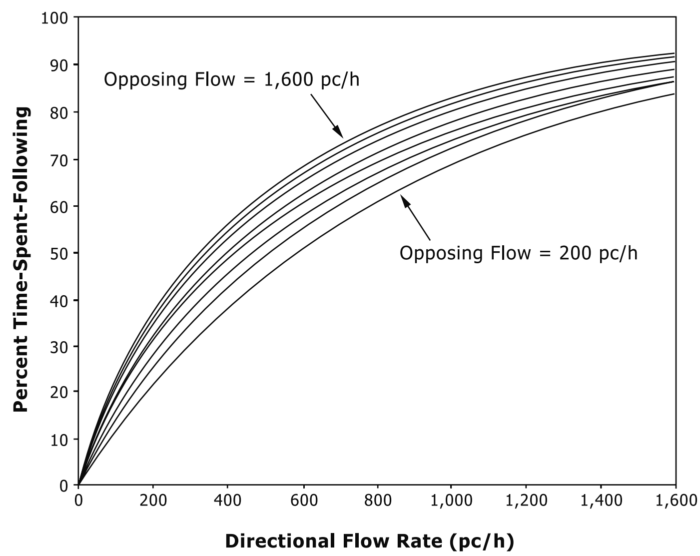
Exhibit 15-2
Speed–Flow and PTSF Relationships for Directional Segments with Base Conditions

LIVE GRAPH
Click here to view

LIVE GRAPH
Click here to view



(a) ATS Versus Directional Flow Rate



(b) PTSF Versus Directional Flow Rate

Capacity is important for evacuation and special event planning.

However, estimation of capacity conditions is important for evacuation planning, special event planning, and evaluation of the downstream impacts of incident bottlenecks once cleared.

Two-way flow rates as high as 3,400 pc/h can be observed for short segments fed by high demands from multiple or multilane facilities. This may occur at tunnels or bridges, for example, but such flow rates cannot be expected over extended segments.

Capacity is not defined for bicycles on two-lane highways because of lack of data. Bicycle volumes approaching capacity do not often occur on two-lane highways except during special bicycle events, and little information is available on which to base a definition.

Levels of Service

Automobile Mode

Because of the wide range of situations in which two-lane highways are found, three measures of effectiveness are incorporated into the methodology of this chapter to determine automobile LOS.

1. *ATS* reflects mobility on a two-lane highway. It is defined as the highway segment length divided by the average travel time taken by vehicles to traverse it during a designated time interval.
2. *PTSF* represents the freedom to maneuver and the comfort and convenience of travel. It is the average percentage of time that vehicles must travel in platoons behind slower vehicles due to the inability to pass. Because this characteristic is difficult to measure in the field, a surrogate measure is the percentage of vehicles traveling at headways of less than 3.0 s at a representative location within the highway segment. PTSF also represents the approximate percentage of vehicles traveling in platoons.
3. *Percent of free-flow speed* (*PFFS*) represents the ability of vehicles to travel at or near the posted speed limit.

On Class I two-lane highways, speed and delay due to passing restrictions are both important to motorists. Therefore, on these highways, LOS is defined in terms of both ATS and PTSF. On Class II highways, travel speed is not a significant issue to drivers. Therefore, on these highways, LOS is defined in terms of PTSF only. On Class III highways, high speeds are not expected. Because the length of Class III segments is generally limited, passing restrictions are also not a major concern. In these cases, drivers would like to make steady progress at or near the speed limit. Therefore, on these highways, PFFS is used to define LOS. The LOS criteria for two-lane highways are shown in Exhibit 15-3.

Exhibit 15-3 Automobile LOS for Two-Lane Highways

LOS	Class I Highways ATS (mi/h)	Class I Highways PTSF (%)	Class II Highways PTSF (%)	Class III Highways PFFS (%)
A	>55	≤35	≤40	>91.7
B	>50–55	>35–50	>40–55	>83.3–91.7
C	>45–50	>50–65	>55–70	>75.0–83.3
D	>40–45	>65–80	>70–85	>66.7–75.0
E	≤40	>80	>85	≤66.7

Because driver expectations and operating characteristics on the three categories of two-lane highways are quite different, it is difficult to provide a single definition of operating conditions at each LOS.

Two characteristics, however, have a significant impact on actual operations and driver perceptions of service:

- *Passing capacity*: Since passing maneuvers on two-lane highways are made in the opposing direction of flow, the ability to pass is limited by the opposing flow rate and by the distribution of gaps in the opposing flow.
- *Passing demand*: As platooning and PTSF increase in a given direction, the demand for passing maneuvers increases. As more drivers are caught in a

platoon behind a slow-moving vehicle, they will desire to make more passing maneuvers.

Both passing capacity and passing demand are related to flow rates. If flow in both directions increases, a difficult trend is established: as passing demand increases, passing capacity decreases.

At LOS A, motorists experience high operating speeds on Class I highways and little difficulty in passing. Platoons of three or more vehicles are rare. On Class II highways, speed would be controlled primarily by roadway conditions. A small amount of platooning would be expected. On Class III highways, drivers should be able to maintain operating speeds close or equal to the free-flow speed (FFS) of the facility.

At LOS B, passing demand and passing capacity are balanced. On both Class I and Class II highways, the degree of platooning becomes noticeable. Some speed reductions are present on Class I highways. On Class III highways, it becomes difficult to maintain FFS operation, but the speed reduction is still relatively small.

At LOS C, most vehicles are traveling in platoons. Speeds are noticeably curtailed on all three classes of highway.

At LOS D, platooning increases significantly. Passing demand is high on both Class I and II facilities, but passing capacity approaches zero. A high percentage vehicles are now traveling in platoons, and PTSF is quite noticeable. On Class III highways, the fall-off from FFS is now significant.

At LOS E, demand is approaching capacity. Passing on Class I and II highways is virtually impossible, and PTSF is more than 80%. Speeds are seriously curtailed. On Class III highways, speed is less than two-thirds the FFS. The lower limit of this LOS represents capacity.

LOS F exists whenever demand flow in one or both directions exceeds the capacity of the segment. Operating conditions are unstable, and heavy congestion exists on all classes of two-lane highway.

Bicycle Mode

Bicycle LOS is based on a traveler-perception model.

Bicycle levels of service for two-lane highway segments are based on a bicycle LOS (BLOS) score, which is in turn based on a traveler-perception model. This score is based, in order of importance, on five variables:

- Average effective width of the outside through lane,
- Motorized vehicle volumes,
- Motorized vehicle speeds,
- Heavy vehicle (truck) volumes, and
- Pavement condition.

The LOS ranges for bicycles on two-lane highways are given in Exhibit 15-4. The same LOS score is used for multilane highways, as described in Chapter 14.

Exhibit 15-4
Bicycle LOS for Two-Lane Highways

LOS	BLOS Score
A	≤1.5
B	>1.5–2.5
C	>2.5–3.5
D	>3.5–4.5
E	>4.5–5.5
F	>5.5

REQUIRED INPUT DATA AND DEFAULT VALUES

Exhibit 15-5 lists the information necessary to apply the methodology. It also contains suggested default values for use when segment-specific information is not available. The user is cautioned, however, that every use of a default value instead of a field-measured, segment-specific variable makes the analysis results more approximate and less related to the specific conditions that describe the study site. Defaults should be used only when field measurements cannot be collected.

Exhibit 15-5
Required Input Data and Default Values for Two-Lane Highways

Required Data	Recommended Default Value	Relevant Modes
	Geometric Data	
Highway class	Must select as appropriate	Auto
Lane width	12 ft	Auto, bicycle
Shoulder width	6 ft	Auto, bicycle
Access-point density (one side)	Classes I and II: 8/mi, Class III: 16/mi	Auto
Terrain	Level or rolling	Auto
Percent no-passing zone[a]	Level: 20%, rolling: 40%, more extreme: 80%	Auto
Speed limit	Speed limit	Bicycle
Base design speed	Speed limit + 10 mi/h	Auto
Length of passing lane (if present)	Must be site-specific	Auto
Pavement condition	4 on FHWA 5-point rating scale (good)	Bicycle
	Demand Data	
Hourly automobile volume	Must be site-specific	Auto, bicycle
Length of analysis period	15 min (0.25 h)	Auto, bicycle
Peak hour factor	0.88	Auto, bicycle
Directional split	60/40	Auto, bicycle
Heavy vehicle percentage[b]	6% trucks	Auto, bicycle
Percent occupied on-highway parking	0%	Bicycle

Notes: [a] Percent no-passing zone may be different in each direction.
[b] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.

The use of some default values is less problematic than others. Lane and shoulder widths of 12 and 6 ft, respectively, are common, particularly on Class I highways. However, these variables have large impacts on bicycle LOS, increasing the importance of segment-specific data. A general assessment of terrain is usually straightforward and requires only general knowledge of the area through which the highway is built. Access-point densities are more difficult and tend to vary widely on a site-by-site basis. Estimating the percent no-passing zones on the basis of a generalized assessment of terrain is also challenging, since the details of vertical and horizontal alignment can have a significant impact on this factor.

FFS is best measured at the site or at a similar site. While adjustments to a base free-flow speed (BFFS) are provided as part of the methodology, no firm guidance on determining the BFFS is given. The default suggestions of Exhibit 15-5 are highly approximate.

In terms of demand data, the length of the analysis period is a recommended HCM standard of 15 min (although longer periods can be examined). The peak hour factor (PHF) is typical but could vary significantly on the basis of localized trip generation characteristics. The directional split is best observed directly, since it can vary widely over time, even at the same location. The recommended default for heavy vehicle presence is also highly approximate. This factor varies widely with local conditions; Chapter 26, Freeway and Highway Segments: Supplemental, provides state-specific default values (*4*).

As is the case with all default values, these values should be used with care, and only when site-specific data cannot be acquired by any reasonable means.

DEMAND VOLUMES AND FLOW RATES

Demand volumes are generally stated in vehicles per hour under prevailing conditions. They are converted in the methodology to demand flow rates in passenger cars per hour under base conditions. The PHF, in particular, is used to convert hourly volumes to flow rates.

If demand volumes are measured in 15-min increments, use of the PHF to convert to flow rates is unnecessary. The worst 15-min period is selected, and flow rates are the 15-min volumes multiplied by 4. When this is done, the PHF is set at 1.00 for the rest of the application.

In measuring demand volumes or flow rates, flow may be restricted by upstream bottlenecks or even signals that are more than 2 mi away from the study site (if they are closer, this methodology is not applicable). Downstream congestion may also affect flows in a study segment. Insofar as is possible, demand volumes and flow rates should reflect the situation that would exist with no upstream or downstream limiting factors.

2. METHODOLOGY

This section presents the details of the methodology for two-lane highways and documents its use in planning and operational analysis applications.

SCOPE OF THE METHODOLOGY

This chapter presents an operational analysis methodology for directional segments of two-lane highways for automobiles and bicyclists. Both directions may be analyzed separately on the facility or segment to obtain a full estimate of operating conditions.

This chapter's automobile methodology addresses the analysis of

- Directional segments in general terrain (level or rolling),
- Directional segments on specific grades, and
- Directional segments including passing and truck climbing lanes.

All segments in mountainous terrain, and all grades of 3% or more that cover a length of 0.6 mi or more, must be analyzed as specific grades.

The methodology is most directly used to determine the LOS on a uniform directional segment of two-lane highway by estimating the measures of effectiveness that define LOS (ATS, PTSF, PFFS). Such an analysis can also be used to determine the capacity of the directional segment or the service flow rate that can be accommodated at any given LOS.

This chapter includes an appendix that addresses specialized treatments for two-lane highways that cannot be evaluated with the basic methodology. Special procedures are also provided to determine the impact of passing lanes or truck climbing lanes in two-lane highway segments.

LIMITATIONS OF THE METHODOLOGY

The operational analysis methodologies in this chapter do not address two-lane highways with signalized intersections. Isolated signalized intersections on two-lane highways may be evaluated with the methodology of Chapter 18, Signalized Intersections. Two-lane highways in urban and suburban areas with multiple signalized intersections 2 mi or less apart should be analyzed as urban streets or arterials with the methodology of Chapter 17, Urban Street Segments.

The bicycle methodology was developed with data collected on urban and suburban streets, including facilities that would be defined as suburban two-lane highways. Although the methodology has been successfully applied to rural two-lane highways in different parts of the United States, users should be aware that conditions on many rural two-lane highways will be outside the range of values used to develop the bicycle LOS model. The ranges of values used in the development of the bicycle LOS model (*5*) are shown below:

- Width of the outside through lane: 10 to 16 ft;
- Shoulder width: 0 to 6 ft;

- Motorized vehicle volumes: up to 36,000 annual average daily traffic (AADT);
- Posted speed: 45 to 50 mi/h;
- Heavy vehicle percentage: 0% to 2%; and
- Pavement condition: 1 to 5 on the Federal Highway Administration (FHWA) 5-point pavement rating scale.

The bicycle LOS methodology also does not take differences in prevalent driver behavior into consideration, although driver behavior may vary considerably both regionally and by facility. In particular, the likelihood of drivers slowing down or providing additional horizontal clearance while passing cyclists plays a significant role in the perceived quality of service of a facility.

AUTOMOBILE MODE

Overview

Exhibit 15-6 illustrates the basic steps in the methodology for two-lane highways. Because the three classes of highways use different service measures to determine LOS, not all steps are applied to each class of facility.

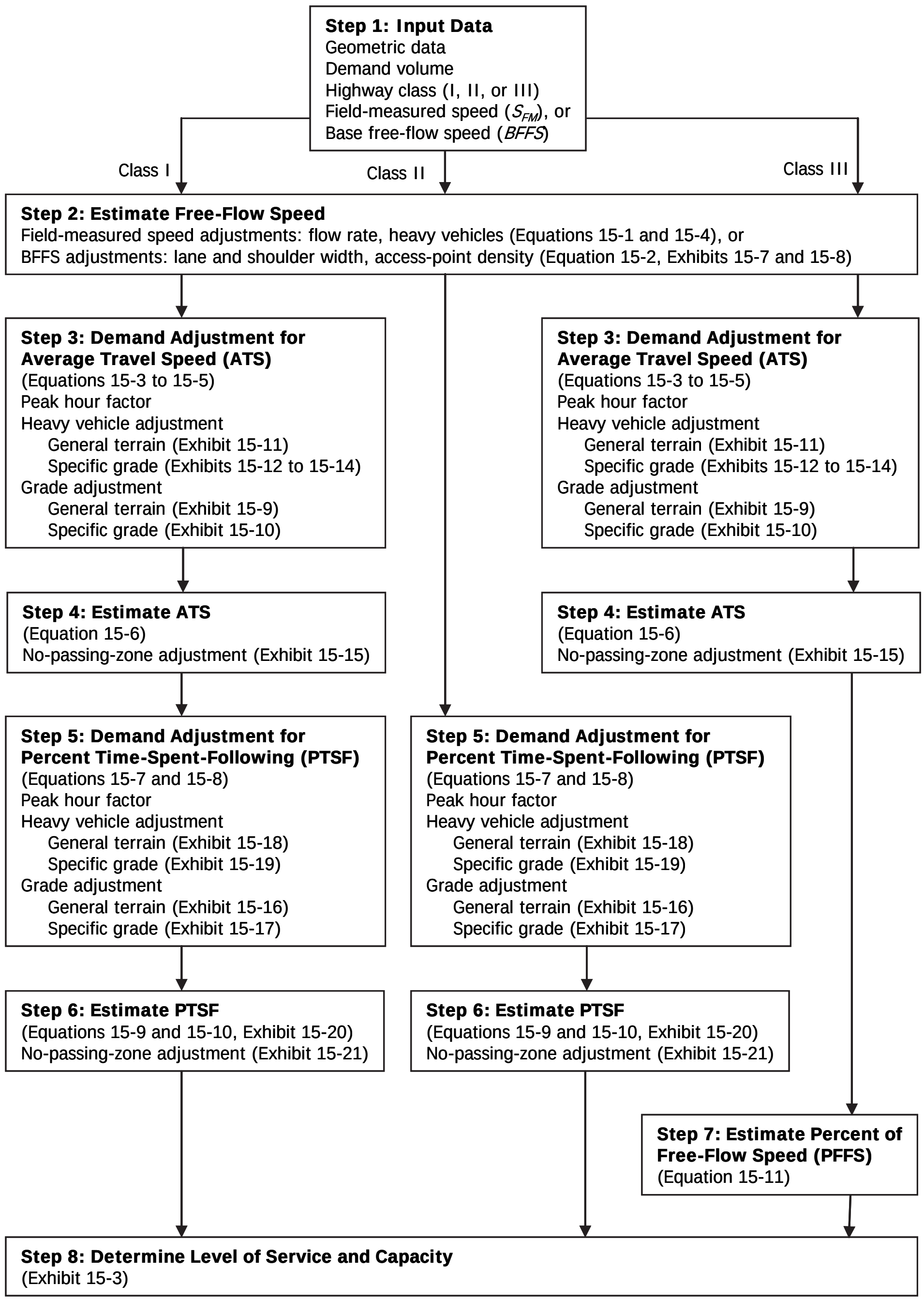
Note that the computational step for estimating ATS applies only to Class I and Class III highways, while the step for estimating PTSF applies only to Class I and Class II highways. The step for estimating PFFS applies only to Class III highways.

Segments for Analysis

The methodology of this chapter applies to uniform directional segments of two-lane highway. While the two directions of flow interact through passing maneuvers (and limitations on passing maneuvers), each direction must be analyzed separately.

Uniform segments have the same or similar traffic and roadway conditions. Segment boundaries should be established at points where a change occurs in any of the following: terrain, lane widths or shoulder width, facility classification, or demand flow rate.

Exhibit 15-6
Flowchart of the Two-Lane Highway Methodology



Computational Steps

Step 1: Input Data

Exhibit 15-5 lists the information that must be available before a two-lane highway segment can be analyzed. The exhibit also lists default values suggested for use when site-specific data are not available.

Step 2: Estimate the FFS

A key step in the analysis of a two-lane highway is the determination of the FFS for the segment. There are three ways to estimate FFS.

Direct Field Measurement

Direct field measurement on the subject highway segment is preferred. Measurements should be taken only in the direction under analysis; if both directions are to be analyzed, then separate measurements in each direction are made. Each directional measurement should be based on a random sample of at least 100 vehicle speeds. The FFS can be directly measured as the mean speed under low-demand conditions (i.e., the two-way flow rate is less than or equal to 200 veh/h).

If the analysis segment cannot be directly observed, then measurements from a similar facility (same highway class, same speed limit, similar environment, etc.) may be used.

Field Measurements at Higher Flow Rates

For some highways, it may be difficult or impossible to observe total flow rates less than 200 veh/h. In such cases, a speed sample may be taken at higher flow rates and adjusted accordingly. The same sampling approach is taken: each direction is separately observed, with each directional sample including at least 100 observed speeds. The measured mean speed is then adjusted with Equation 15-1:

Equation 15-1

$$FFS = S_{FM} + 0.00776\left(\frac{v}{f_{HV,ATS}}\right)$$

where

FFS = free-flow speed (mi/h);

S_{FM} = mean speed of sample ($v > 200$ veh/h) (mi/h);

v = total demand flow rate, both directions, during period of speed measurements (veh/h); and

$f_{HV,ATS}$ = heavy vehicle adjustment factor for ATS, from Equation 15-4 or Equation 15-5.

Estimating FFS

FFS on two-lane highways ranges from 45 mi/h to as high as 70 mi/h. BFFS reflects alignment of the facility and the nature of traffic.

The FFS can be estimated indirectly if field data are not available. This is a greater challenge on two-lane highways than on other types of uninterrupted-flow facilities. FFS on two-lane highways covers a significant range, from as low as 45 mi/h to as high as 70 mi/h. To estimate the FFS, the analyst must characterize the operating conditions of the facility in terms of a BFFS that reflects the nature of the traffic and the alignment of the facility. Unfortunately, because of the broad range of speeds that occur and the importance of local and regional factors that influence driver-desired speeds, little guidance on estimating the BFFS can be given.

Estimates of BFFS can be developed on the basis of speed data and local knowledge of operating conditions on similar facilities. As will be seen, once the BFFS is determined, adjustments for lane and shoulder widths and for the density of unsignalized access points are applied to estimate the FFS. In concept, the BFFS is the speed that would be expected on the basis of the facility's horizontal and vertical alignment, if standard lane and shoulder widths were present and there were no roadside access points. Thus, the *design speed* of the facility might be an acceptable estimator of BFFS, since it is based primarily on horizontal and vertical alignment. Posted speed limits may not reflect current conditions or driver desires. A rough estimate of BFFS might be taken as the posted speed limit plus 10 mi/h.

Once a BFFS is determined, the actual FFS may be estimated as follows:

Equation 15-2

$$FFS = BFFS - f_{LS} - f_A$$

where

FFS = free-flow speed (mi/h),

$BFFS$ = base free-flow speed (mi/h),

f_{LS} = adjustment for lane and shoulder width (mi/h), and

f_A = adjustment for access-point density (mi/h).

When field measurements are used to estimate FFS, standard approaches and sampling techniques should be applied. Guidance on field speed studies is provided in standard traffic engineering texts and elsewhere (*3*).

Adjustment factors for use in Equation 15-2 are found in Exhibit 15-7 (lane and shoulder width) and Exhibit 15-8 (access-point density).

Exhibit 15-7
Adjustment Factor for Lane and Shoulder Width (f_{LS})

Lane Width (ft)	Shoulder Width (ft)			
	≥0 <2	≥2 <4	≥4 <6	≥6
≥9 <10	6.4	4.8	3.5	2.2
≥10 <11	5.3	3.7	2.4	1.1
≥ 11 <12	4.7	3.0	1.7	0.4
≥12	4.2	2.6	1.3	0.0

Exhibit 15-8
Adjustment Factor for Access-Point Density (f_A)

Access Points per Mile (Two Directions)	Reduction in FFS (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
40	10.0

Note: Interpolation to the nearest 0.1 is recommended.

The access-point density is computed by dividing the total number of unsignalized intersections and driveways on *both* sides of the roadway segment by the length of the segment (in miles). Thus, in analyzing the two directions of the highway and estimating the FFS, the FFS will be the same in *both* directions. If the FFS is measured in the field, the value could be different in each direction.

If a highway contains sharp horizontal curves with design speeds substantially below those of the rest of the segment, it may be desirable to

determine the FFS separately for curves and tangents and to compute a weighted-average FFS for the segment as a whole.

The data for FFS relationships in this chapter include both commuter and noncommuter traffic. There were no significant differences between the two. However, it is expected that commuters and other regular users will use a facility more efficiently than recreational and other occasional users. If the effect of driver population is a concern, the FFS should be measured in the field.

Step 3: Demand Adjustment for ATS

This computational step is applied only in cases of Class I and Class III two-lane highways. LOS on Class II highways is not based on ATS, and therefore this step is skipped for those highways.

Demand volumes in both directions (analysis direction and opposing direction) must be converted to flow rates under equivalent base conditions with Equation 15-3:

Equation 15-3

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

where

$v_{i,ATS}$ = demand flow rate *i* for ATS estimation (pc/h);

i = "d" (analysis direction) or "o" (opposing direction);

V_i = demand volume for direction *i* (veh/h);

$f_{g,ATS}$ = grade adjustment factor, from Exhibit 15-9 or Exhibit 15-10; and

$f_{HV,ATS}$ = heavy vehicle adjustment factor, from Equation 15-4 or Equation 15-5.

PHF

The PHF represents the variation in traffic flow within the hour. Two-lane highway analysis is based on the demand flow rates for a peak 15-min period within the analysis hour—usually (but not necessarily) the peak hour. If flow rates for the peak 15 min have been directly measured, the PHF used in Equation 15-3 is set equal to 1.00.

ATS Grade Adjustment Factor

The grade adjustment factor $f_{g,ATS}$ depends on the terrain. Factors are defined for

- Extended segments (≥2 mi) of level terrain,
- Extended segments (≥2 mi) of rolling terrain,
- Specific upgrades, and
- Specific downgrades.

Any grade of 3% or steeper and 0.6 mi or longer *must* be analyzed as a specific upgrade or downgrade, depending on the analysis direction being considered. However, a grade of 3% or more *may* be analyzed as a specific grade if it is 0.25 mi or longer.

Exhibit 15-9 shows grade adjustment factors for extended segments of level and rolling terrain, as well as for specific downgrades. Exhibit 15-9 is entered with the one-direction demand flow rate v_{vph}, in vehicles per hour.

Exhibit 15-9
ATS Grade Adjustment Factor ($f_{g,ATS}$) for Level Terrain, Rolling Terrain, and Specific Downgrades

One-Direction Demand Flow Rate, v_{vph} (veh/h)	Adjustment Factor: Level Terrain and Specific Downgrades	Adjustment Factor: Rolling Terrain
≤100	1.00	0.67
200	1.00	0.75
300	1.00	0.83
400	1.00	0.90
500	1.00	0.95
600	1.00	0.97
700	1.00	0.98
800	1.00	0.99
≥900	1.00	1.00

Note: Interpolation to the nearest 0.01 is recommended.

If demand is expressed as an hourly volume, it must be divided by the PHF ($v_{vph} = V/PHF$) to obtain the appropriate factor. Other adjustment factor tables associated with Equation 15-3 are entered with this value as well.

Note that the adjustment factor for level terrain is 1.00, since level terrain is one of the base conditions. For the purposes of grade adjustment, specific downgrade segments are treated as level terrain.

Exhibit 15-10 shows grade adjustment factors for specific upgrades. The negative impact of upgrades on two-lane highway speeds increases as both the severity of the upgrade and its length increase. The impact, however, declines as demand flow rate increases. At higher demand flow rates, lower speeds would already result, and the additional impact of the upgrades is less severe.

ATS Heavy Vehicle Adjustment Factor

The base conditions for two-lane highways include 100% passenger cars in the traffic stream. This is a rare occurrence, and the presence of heavy vehicles in the traffic stream reduces the ATS.

In general, a heavy vehicle is defined as any vehicle (or vehicle–trailer unit) with more than four wheels on the ground during normal operation. Heavy vehicles are classified as trucks or recreational vehicles (RVs). Trucks cover a wide variety of vehicles from small pickup and panel trucks with more than four wheels to double and triple tractor–trailer units. Small pickup and panel trucks with only four wheels are classified as passenger cars. All school, transit, or intercity buses are classified as trucks. The RV classification also covers a wide range of vehicles, including motorized campers, motor homes, and cars or small trucks that are towing trailers.

Exhibit 15-10
ATS Grade Adjustment Factor ($f_{g,ATS}$) for Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h) ≤100	200	300	400	500	600	700	800	≥900
≥3 <3.5	0.25	0.78	0.84	0.87	0.91	1.00	1.00	1.00	1.00	1.00
	0.50	0.75	0.83	0.86	0.90	1.00	1.00	1.00	1.00	1.00
	0.75	0.73	0.81	0.85	0.89	1.00	1.00	1.00	1.00	1.00
	1.00	0.73	0.79	0.83	0.88	1.00	1.00	1.00	1.00	1.00
	1.50	0.73	0.79	0.83	0.87	0.99	0.99	1.00	1.00	1.00
	2.00	0.73	0.79	0.82	0.86	0.98	0.98	0.99	1.00	1.00
	3.00	0.73	0.78	0.82	0.85	0.95	0.96	0.96	0.97	0.98
	≥4.00	0.73	0.78	0.81	0.85	0.94	0.94	0.95	0.95	0.96
≥3.5 <4.5	0.25	0.75	0.83	0.86	0.90	1.00	1.00	1.00	1.00	1.00
	0.50	0.72	0.80	0.84	0.88	1.00	1.00	1.00	1.00	1.00
	0.75	0.67	0.77	0.81	0.86	1.00	1.00	1.00	1.00	1.00
	1.00	0.65	0.73	0.77	0.81	0.94	0.95	0.97	1.00	1.00
	1.50	0.63	0.72	0.76	0.80	0.93	0.95	0.96	1.00	1.00
	2.00	0.62	0.70	0.74	0.79	0.93	0.94	0.96	1.00	1.00
	3.00	0.61	0.69	0.74	0.78	0.92	0.93	0.94	0.98	1.00
	≥4.00	0.61	0.69	0.73	0.78	0.91	0.91	0.92	0.96	1.00
≥4.5 <5.5	0.25	0.71	0.79	0.83	0.88	1.00	1.00	1.00	1.00	1.00
	0.50	0.60	0.70	0.74	0.79	0.94	0.95	0.97	1.00	1.00
	0.75	0.55	0.65	0.70	0.75	0.91	0.93	0.95	1.00	1.00
	1.00	0.54	0.64	0.69	0.74	0.91	0.93	0.95	1.00	1.00
	1.50	0.52	0.62	0.67	0.72	0.88	0.90	0.93	1.00	1.00
	2.00	0.51	0.61	0.66	0.71	0.87	0.89	0.92	0.99	1.00
	3.00	0.51	0.61	0.65	0.70	0.86	0.88	0.91	0.98	0.99
	≥4.00	0.51	0.60	0.65	0.69	0.84	0.86	0.88	0.95	0.97
≥5.5 <6.5	0.25	0.57	0.68	0.72	0.77	0.93	0.94	0.96	1.00	1.00
	0.50	0.52	0.62	0.66	0.71	0.87	0.90	0.92	1.00	1.00
	0.75	0.49	0.57	0.62	0.68	0.85	0.88	0.90	1.00	1.00
	1.00	0.46	0.56	0.60	0.65	0.82	0.85	0.88	1.00	1.00
	1.50	0.44	0.54	0.59	0.64	0.81	0.84	0.87	0.98	1.00
	2.00	0.43	0.53	0.58	0.63	0.81	0.83	0.86	0.97	0.99
	3.00	0.41	0.51	0.56	0.61	0.79	0.82	0.85	0.97	0.99
	≥4.00	0.40	0.50	0.55	0.61	0.79	0.82	0.85	0.97	0.99
≥6.5	0.25	0.54	0.64	0.68	0.73	0.88	0.90	0.92	1.00	1.00
	0.50	0.43	0.53	0.57	0.62	0.79	0.82	0.85	0.98	1.00
	0.75	0.39	0.49	0.54	0.59	0.77	0.80	0.83	0.96	1.00
	1.00	0.37	0.45	0.50	0.54	0.74	0.77	0.81	0.96	1.00
	1.50	0.35	0.45	0.49	0.54	0.71	0.75	0.79	0.96	1.00
	2.00	0.34	0.44	0.48	0.53	0.71	0.74	0.78	0.94	0.99
	3.00	0.34	0.44	0.48	0.53	0.70	0.73	0.77	0.93	0.98
	≥4.00	0.33	0.43	0.47	0.52	0.70	0.73	0.77	0.91	0.95

Note: Straight-line interpolation of $f_{g,ATS}$ for length of grade and demand flow permitted to the nearest 0.01.

Exhibit 15-11
ATS Passenger Car Equivalents for Trucks (E_T) and RVs (E_R) for Level Terrain, Rolling Terrain, and Specific Downgrades

Vehicle Type	Directional Demand Flow Rate, v_{vph} (veh/h)	Level Terrain and Specific Downgrades	Rolling Terrain
Trucks, E_T	≤100	1.9	2.7
	200	1.5	2.3
	300	1.4	2.1
	400	1.3	2.0
	500	1.2	1.8
	600	1.1	1.7
	700	1.1	1.6
	800	1.1	1.4
	≥900	1.0	1.3
RVs, E_R	All flows	1.0	1.1

Note: Interpolation to the nearest 0.1 is recommended.

Determining the heavy vehicle adjustment factor is a two-step process:

1. Passenger car equivalents are found for trucks (E_T) and RVs (E_R) under prevailing conditions.
2. A heavy vehicle adjustment factor is computed from the passenger car equivalents with Equation 15-4:

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Equation 15-4

where

$f_{HV,ATS}$ = heavy vehicle adjustment factor for ATS estimation,

P_T = proportion of trucks in the traffic stream (decimal),

P_R = proportion of RVs in the traffic stream (decimal),

E_T = passenger car equivalent for trucks from Exhibit 15-11 or Exhibit 15-12, and

E_R = passenger car equivalent for RVs from Exhibit 15-11 or Exhibit 15-13.

The passenger car equivalent is the number of passenger cars displaced from the traffic stream by one truck or RV. Passenger car equivalents are defined for several situations:

- Extended sections of general level or rolling terrain,
- Specific upgrades, and
- Specific downgrades.

Exhibit 15-11 contains passenger car equivalents for trucks and RVs in general terrain segments and for specific downgrades, which are treated as level terrain in most cases. A special procedure is provided in the next section to evaluate specific downgrades on which significant numbers of trucks must reduce their speed to crawl speed to maintain control.

Exhibit 15-12 and Exhibit 15-13 show passenger car equivalents for trucks and RVs, respectively, on specific upgrades.

ATS Passenger Car Equivalents for Specific Downgrades Where Trucks Travel at Crawl Speed

As noted previously, any downgrade of 3% or more *and* 0.6 mi or longer must be analyzed as a specific downgrade. If the slope of the downgrade varies, it should be analyzed as a single composite by using an average grade computed by dividing the total change in elevation by the total length of grade and expressing the result as a percentage.

Most specific downgrades will be treated as level terrain for analysis purposes. Some downgrades, however, are severe enough to force some trucks into crawl speed. In such cases, the truck drivers are forced to operate in a low gear to apply engine braking, since the normal brake system would not be sufficient to slow or stop a heavy vehicle from gaining too much momentum as it travels down a sharp downgrade. There are no general guidelines for identifying when or where these situations will occur, other than direct observation of heavy vehicle operations.

Exhibit 15-12
ATS Passenger Car Equivalents for Trucks (E_T) on Specific Upgrades

Grade (%)	Grade Length (mi)	≤100	200	300	400	500	600	700	800	≥900
		Directional Demand Flow Rate, v_{vph} (veh/h)								
≥3 <3.5	0.25	2.6	2.4	2.3	2.2	1.8	1.8	1.7	1.3	1.1
	0.50	3.7	3.4	3.3	3.2	2.7	2.6	2.6	2.3	2.0
	0.75	4.6	4.4	4.3	4.2	3.7	3.6	3.4	2.4	1.9
	1.00	5.2	5.0	4.9	4.9	4.4	4.2	4.1	3.0	1.6
	1.50	6.2	6.0	5.9	5.8	5.3	5.0	4.8	3.6	2.9
	2.00	7.3	6.9	6.7	6.5	5.7	5.5	5.3	4.1	3.5
	3.00	8.4	8.0	7.7	7.5	6.5	6.2	6.0	4.6	3.9
	≥4.00	9.4	8.8	8.6	8.3	7.2	6.9	6.6	4.8	3.7
≥3.5 <4.5	0.25	3.8	3.4	3.2	3.0	2.3	2.2	2.2	1.7	1.5
	0.50	5.5	5.3	5.1	5.0	4.4	4.2	4.0	2.8	2.2
	0.75	6.5	6.4	6.5	6.5	6.3	5.9	5.6	3.6	2.6
	1.00	7.9	7.6	7.4	7.3	6.7	6.6	6.4	5.3	4.7
	1.50	9.6	9.2	9.0	8.9	8.1	7.9	7.7	6.5	5.9
	2.00	10.3	10.1	10.0	9.9	9.4	9.1	8.9	7.4	6.7
	3.00	11.4	11.3	11.2	11.2	10.7	10.3	10.0	8.0	7.0
	≥4.00	12.4	12.2	12.2	12.1	11.5	11.2	10.8	8.6	7.5
≥4.5 <5.5	0.25	4.4	4.0	3.7	3.5	2.7	2.7	2.7	2.6	2.5
	0.50	6.0	6.0	6.0	6.0	5.9	5.7	5.6	4.6	4.2
	0.75	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	1.00	9.2	9.2	9.1	9.1	9.0	9.0	9.0	8.9	8.8
	1.50	10.6	10.6	10.6	10.6	10.5	10.4	10.4	10.2	10.1
	2.00	11.8	11.8	11.8	11.8	11.6	11.6	11.5	11.1	10.9
	3.00	13.7	13.7	13.6	13.6	13.3	13.1	13.0	11.9	11.3
	≥4.00	15.3	15.3	15.2	15.2	14.6	14.2	13.8	11.3	10.0
≥5.5 <6.5	0.25	4.8	4.6	4.5	4.4	4.0	3.9	3.8	3.2	2.9
	0.50	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
	0.75	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1
	1.00	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.2	10.1
	1.50	11.9	11.9	11.9	11.9	11.8	11.8	11.8	11.7	11.6
	2.00	12.8	12.8	12.8	12.8	12.7	12.7	12.7	12.6	12.5
	3.00	14.4	14.4	14.4	14.4	14.3	14.3	14.3	14.2	14.1
	≥4.00	15.4	15.4	15.3	15.3	15.2	15.1	15.1	14.9	14.8
≥6.5	0.25	5.1	5.1	5.0	5.0	4.8	4.7	4.7	4.5	4.4
	0.50	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8
	0.75	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8
	1.00	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.3	10.2
	1.50	12.0	12.0	12.0	12.0	11.9	11.9	11.9	11.8	11.7
	2.00	12.9	12.9	12.9	12.9	12.8	12.8	12.8	12.7	12.6
	3.00	14.5	14.5	14.5	14.5	14.4	14.4	14.4	14.3	14.2
	≥4.00	15.4	15.4	15.4	15.4	15.3	15.3	15.3	15.2	15.1

Note: Interpolation for length of grade and demand flow rate to the nearest 0.1 is recommended.

Exhibit 15-13
ATS Passenger Car Equivalents for RVs (E_R) on Specific Upgrades

Grade (%)	Grade Length (mi)	≤100	200	300	400	500	600	700	800	≥900
		Directional Demand Flow Rate, v_{vph} (veh/h)								
≥3 <3.5	≤0.25	1.1	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0
	>0.25 ≤0.75	1.2	1.2	1.1	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75 ≤1.25	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>1.25 ≤2.25	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>2.25	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
≥3.5 <4.5	≤0.75	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75 ≤3.50	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>3.50	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
≥4.5 <5.5	≤2.50	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
≥5.5 <6.5	≤0.75	1.5	1.4	1.3	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75 ≤2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50 ≤3.50	1.6	1.5	1.4	1.3	1.2	1.1	1.0	1.0	1.0
	>3.50	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.2	1.1
≥6.5	≤2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50 ≤3.50	1.6	1.5	1.4	1.2	1.3	1.3	1.3	1.3	1.3
	>3.50	1.6	1.6	1.6	1.5	1.5	1.5	1.4	1.4	1.4

Note: Interpolation in this exhibit is not recommended.

When this situation exists, the heavy vehicle adjustment factor $f_{HV,ATS}$ is found with Equation 15-5 instead of Equation 15-4:

$$f_{HV,ATS} = \frac{1}{1 + P_{TC} \times P_T(E_{TC} - 1) + (1 - P_{TC}) \times P_T \times (E_T - 1) + P_R(E_R - 1)}$$

Equation 15-5

where

P_{TC} = proportion of trucks operating at crawl speed (decimal); and

E_{TC} = passenger car equivalent for trucks operating at crawl speed, from Exhibit 15-14.

All other variables are as previously defined. Note that P_{TC} is the flow rate of trucks traveling at crawl speed divided by the flow rate of all trucks.

Exhibit 15-14
ATS Passenger Car Equivalents (E_{TC}) for Trucks on Downgrades Traveling at Crawl Speed

Difference Between FFS and Truck Crawl Speed (mi/h)	Directional Demand Flow Rate, v_{vph} (veh/h)								
	≤100	200	300	400	500	600	700	800	≥900
≤15	4.7	4.1	3.6	3.1	2.6	2.1	1.6	1.0	1.0
20	9.9	8.7	7.8	6.7	5.8	4.9	4.0	2.7	1.0
25	15.1	13.5	12.0	10.4	9.0	7.7	6.4	5.1	3.8
30	22.0	19.8	17.5	15.6	13.1	11.6	9.2	6.1	4.1
35	29.0	26.0	23.1	20.1	17.3	14.6	11.9	9.2	6.5
≥40	35.9	32.3	28.6	24.9	21.4	18.1	14.7	11.3	7.9

Note: Interpolation against both speed difference and demand flow rate to the nearest 0.1 is recommended.

Step 4: Estimate the ATS

As was the case with Step 3, this step applies only to Class I and Class III two-lane highways. Class II highways do not use ATS as a LOS measure.

The ATS is estimated from the FFS, the demand flow rate, the opposing flow rate, and the percentage of no-passing zones in the analysis direction. The ATS is computed from Equation 15-6:

$$ATS_d = FFS - 0.00776(v_{d,ATS} + v_{o,ATS}) - f_{np,ATS}$$

Equation 15-6

where

ATS_d = average travel speed in the analysis direction (mi/h);

FFS = free-flow speed (mi/h);

$v_{d,ATS}$ = demand flow rate for ATS determination in the analysis direction (pc/h);

$v_{o,ATS}$ = demand flow rate for ATS determination in the opposing direction (pc/h); and

$f_{np,ATS}$ = adjustment factor for ATS determination for the percentage of no-passing zones in the analysis direction, from Exhibit 15-15.

Exhibit 15-15
ATS Adjustment Factor for No-Passing Zones ($f_{np,ATS}$)

Opposing Demand Flow Rate, v_o (pc/h)	Percent No-Passing Zones ≤ 20	40	60	80	100
FFS ≥ 65 mi/h					
≤100	1.1	2.2	2.8	3.0	3.1
200	2.2	3.3	3.9	4.0	4.2
400	1.6	2.3	2.7	2.8	2.9
600	1.4	1.5	1.7	1.9	2.0
800	0.7	1.0	1.2	1.4	1.5
1,000	0.6	0.8	1.1	1.1	1.2
1,200	0.6	0.8	0.9	1.0	1.1
1,400	0.6	0.7	0.9	0.9	0.9
≥1,600	0.6	0.7	0.7	0.7	0.8
FFS = 60 mi/h					
≤100	0.7	1.7	2.5	2.8	2.9
200	1.9	2.9	3.7	4.0	4.2
400	1.4	2.0	2.5	2.7	3.9
600	1.1	1.3	1.6	1.9	2.0
800	0.6	0.9	1.1	1.3	1.4
1,000	0.6	0.7	0.9	1.1	1.2
1,200	0.5	0.7	0.9	0.9	1.1
1,400	0.5	0.6	0.8	0.8	0.9
≥1,600	0.5	0.6	0.7	0.7	0.7
FFS = 55 mi/h					
≤100	0.5	1.2	2.2	2.6	2.7
200	1.5	2.4	3.5	3.9	4.1
400	1.3	1.9	2.4	2.7	2.8
600	0.9	1.1	1.6	1.8	1.9
800	0.5	0.7	1.1	1.2	1.4
1,000	0.5	0.6	0.8	0.9	1.1
1,200	0.5	0.6	0.7	0.9	1.0
1,400	0.5	0.6	0.7	0.7	0.9
≥1,600	0.5	0.6	0.6	0.6	0.7
FFS = 50 mi/h					
≤100	0.2	0.7	1.9	2.4	2.5
200	1.2	2.0	3.3	3.9	4.0
400	1.1	1.6	2.2	2.6	2.7
600	0.6	0.9	1.4	1.7	1.9
800	0.4	0.6	0.9	1.2	1.3
1,000	0.4	0.4	0.7	0.9	1.1
1,200	0.4	0.4	0.7	0.8	1.0
1,400	0.4	0.4	0.6	0.7	0.8
≥1,600	0.4	0.4	0.5	0.5	0.5
FFS ≤ 45 mi/h					
≤100	0.1	0.4	1.7	2.2	2.4
200	0.9	1.6	3.1	3.8	4.0
400	0.9	0.5	2.0	2.5	2.7
600	0.4	0.3	1.3	1.7	1.8
800	0.3	0.3	0.8	1.1	1.2
1,000	0.3	0.3	0.6	0.8	1.1
1,200	0.3	0.3	0.6	0.7	1.0
1,400	0.3	0.3	0.6	0.6	0.7
≥1,600	0.3	0.3	0.4	0.4	0.6

Note: Interpolation of $f_{np,ATS}$ for percent no-passing zones, demand flow rate, and FFS to the nearest 0.1 is recommended.

Exhibit 15-15 is entered with v_o in passenger cars per hour, not v_{vph} in vehicles per hour. At this point in the computational process, fully adjusted demand flow rates are available and are used in the determination of *ATS*. As shown in this exhibit, the effect of no-passing zones is greatest when opposing flow rates are low. As opposing flow rates increase, the effect decreases to zero, since passing and no-passing zones become irrelevant when the opposing flow rate allows no opportunities to pass.

Step 5: Demand Adjustment for PTSF

This computational step is applied only in cases of Class I and Class II two-lane highways. LOS on Class III highways is not based on PTSF, and therefore this step is skipped for those highways.

The demand volume adjustment process for estimating PTSF is structurally similar to that for ATS. The general approach is the same, but different adjustment factors are used, and the resulting adjusted flow rates will be different from those used in estimating ATS. Therefore, a detailed discussion of the process is not included here, since it is the same as that described for ATS estimates.

Equation 15-7 and Equation 15-8 are used to determine demand flow rates for the estimation of PTSF:

Equation 15-7

$$v_{i,PTSF} = \frac{V_i}{PHF \times f_{g,PTSF} \times f_{HV,PTSF}}$$

Equation 15-8

$$f_{HV,PTSF} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where

$v_{i,PTSF}$ = demand flow rate i for determination of PTSF (pc/h);

i = "d" (analysis direction) or "o" (opposing direction);

$f_{g,PTSF}$ = grade adjustment factor for PTSF determination, from Exhibit 15-16 or Exhibit 15-17; and

$f_{HV,PTSF}$ = heavy vehicle adjustment factor for PTSF determination, from Exhibit 15-18 or Exhibit 15-19.

All other variables are as previously defined.

PTSF Grade Adjustment Factor

As was the case for the ATS adjustment process, grade adjustment factors are defined for general terrain segments (level or rolling), specific upgrades, and specific downgrades. Exhibit 15-16 gives the adjustment factors for general terrain segments and specific downgrades (which are treated as level terrain). Exhibit 15-17 shows adjustment factors for specific upgrades. These adjustments are used to compute demand flow rates, and the exhibits are again entered with $v_{vph} = V/PHF$.

Exhibit 15-16
PTSF Grade Adjustment Factor ($f_{g,PTSF}$) for Level Terrain, Rolling Terrain, and Specific Downgrades

Directional Demand Flow Rate, v_{vph} (veh/h)	Level Terrain and Specific Downgrades	Rolling Terrain
≤100	1.00	0.73
200	1.00	0.80
300	1.00	0.85
400	1.00	0.90
500	1.00	0.96
600	1.00	0.97
700	1.00	0.99
800	1.00	1.00
≥900	1.00	1.00

Note: Interpolation to the nearest 0.01 is recommended.

Exhibit 15-17
PTSF Grade Adjustment Factor ($f_{g,PTSF}$) for Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h) ≤100	200	300	400	500	600	700	800	≥900
≥3 <3.5	0.25	1.00	0.99	0.97	0.96	0.92	0.92	0.92	0.92	0.92
	0.50	1.00	0.99	0.98	0.97	0.93	0.93	0.93	0.93	0.93
	0.75	1.00	0.99	0.98	0.97	0.93	0.93	0.93	0.93	0.93
	1.00	1.00	0.99	0.98	0.97	0.93	0.93	0.93	0.93	0.93
	1.50	1.00	0.99	0.98	0.97	0.94	0.94	0.94	0.94	0.94
	2.00	1.00	0.99	0.98	0.98	0.95	0.95	0.95	0.95	0.95
	3.00	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.96	0.96
	≥4.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.97	0.97
≥3.5 <4.5	0.25	1.00	0.99	0.98	0.97	0.94	0.93	0.93	0.92	0.92
	0.50	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.96	0.95
	0.75	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.96	0.96
	1.00	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.97	0.97
	1.50	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.97	0.97
	2.00	1.00	1.00	0.99	0.99	0.98	0.98	0.98	0.98	0.98
	3.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	≥4.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
≥4.5 <5.5	0.25	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.97	0.97
	≥0.50	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
≥5.5	All	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Note: Interpolation for length of grade and demand flow rate to the nearest 0.01 is recommended.

PTSF Heavy Vehicle Adjustment Factor

The process for determining the heavy vehicle adjustment factor used in estimating PTSF (Equation 15-8) is similar to that used in estimating ATS. Passenger car equivalents must be found for trucks (E_T) and recreational vehicles (E_R). Equivalents for both trucks and RVs in general terrain segments (level, rolling) and on specific downgrades (which are treated as level terrain) are found in Exhibit 15-18. In estimating PTSF, there is no special procedure for trucks traveling at crawl speed on specific downgrades. Equivalents for trucks and RVs on specific upgrades are found in Exhibit 15-19.

Exhibit 15-18
PTSF Passenger Car Equivalents for Trucks (E_T) and RVs (E_R) for Level Terrain, Rolling Terrain, and Specific Downgrades

Vehicle Type	Directional Demand Flow Rate, v_{vph} (veh/h)	Level and Specific Downgrade	Rolling
Trucks, E_T	≤100	1.1	1.9
	200	1.1	1.8
	300	1.1	1.7
	400	1.1	1.6
	500	1.0	1.4
	600	1.0	1.2
	700	1.0	1.0
	800	1.0	1.0
	≥900	1.0	1.0
RVs, E_R	All	1.0	1.0

Note: Interpolation in this exhibit is not recommended.

Exhibit 15-19
PTSF Passenger Car Equivalents for Trucks (E_T) and RVs (E_R) on Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h) ≤100	200	300	400	500	600	700	800	≥900
Passenger Car Equivalents for Trucks (E_T)										
≥3 <3.5	≤2.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.5	1.3	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	≥4.00	1.6	1.4	1.3	1.3	1.0	1.0	1.0	1.0	1.0
≥3.5 <4.5	≤1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	2.00	1.6	1.3	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.8	1.4	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	≥4.00	2.1	1.9	1.8	1.7	1.4	1.4	1.4	1.4	1.4
≥4.5 <5.5	≤1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	2.00	1.7	1.6	1.6	1.6	1.5	1.4	1.4	1.3	1.3
	3.00	2.4	2.2	2.2	2.1	1.9	1.8	1.8	1.7	1.7
	≥4.00	3.5	3.1	2.9	2.7	2.1	2.0	2.0	1.8	1.8
≥5.5 <6.5	≤0.75	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.00	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2
	1.50	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.6
	2.00	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.8	1.8
	3.00	3.4	3.2	3.0	2.9	2.4	2.3	2.3	1.9	1.9
	≥4.00	4.5	4.1	3.9	3.7	2.9	2.7	2.6	2.0	2.0
≥6.5	≤0.50	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	0.75	1.0	1.0	1.0	1.0	1.1	1.1	1.1	1.0	1.0
	1.00	1.3	1.3	1.3	1.4	1.4	1.5	1.5	1.4	1.4
	1.50	2.1	2.1	2.1	2.1	2.0	2.0	2.0	2.0	2.0
	2.00	2.9	2.8	2.7	2.7	2.4	2.4	2.3	2.3	2.3
	3.00	4.2	3.9	3.7	3.6	3.0	2.8	2.7	2.2	2.2
	≥4.00	5.0	4.6	4.4	4.2	3.3	3.1	2.9	2.7	2.5
Passenger Car Equivalents for RVs (E_R)										
All	All	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Note: Interpolation for length of grade and demand flow rate to the nearest 0.1 is recommended.

Step 6: Estimate the PTSF

This step is only applied to Class I and Class II two-lane highways. Class III highways do not use PTSF to determine LOS.

Once the demand flows for estimating PTSF are computed, the PTSF is estimated with Equation 15-9:

Equation 15-9

$$PTSF_d = BPTSF_d + f_{np,PTSF}\left(\frac{v_{d,PTSF}}{v_{d,PTSF} + v_{o,PTSF}}\right)$$

where

$PTSF_d$ = percent time-spent-following in the analysis direction (decimal);

$BPTSF_d$ = base percent time-spent-following in the analysis direction, from Equation 15-10;

$f_{np,PTSF}$ = adjustment to PTSF for the percentage of no-passing zones in the analysis segment, from Exhibit 15-21;

$v_{d,PTSF}$ = demand flow rate in the analysis direction for estimation of PTSF (pc/h); and

$v_{o,PTSF}$ = demand flow rate in the opposing direction for estimation of PTSF (pc/h).

The base percent time-spent-following (BPTSF) applies to base conditions and is estimated by Equation 15-10:

Equation 15-10

$$BPTSF_d = 100\left[1 - \exp\left(av_d^b\right)\right]$$

where a and b are constants drawn from Exhibit 15-20 and all other terms are as previously defined.

Exhibit 15-20 and Exhibit 15-21 are entered with demand flow rates fully converted to passenger cars per hour under base conditions (v_o and v_d).

Exhibit 15-20
PTSF Coefficients for Use in Equation 15-10 for Estimating BPTSF

Opposing Demand Flow Rate, v_o (pc/h)	Coefficient a	Coefficient b
≤200	-0.0014	0.973
400	-0.0022	0.923
600	-0.0033	0.870
800	-0.0045	0.833
1,000	-0.0049	0.829
1,200	-0.0054	0.825
1,400	-0.0058	0.821
≥1,600	-0.0062	0.817

Note: Straight-line interpolation of a to the nearest 0.0001 and b to the nearest 0.001 is recommended.

Exhibit 15-21
No-Passing-Zone Adjustment Factor ($f_{np,PTSF}$) for Determination of PTSF

Total Two-Way Flow Rate, $v = v_d + v_o$ (pc/h)	Percent No-Passing Zones					
	0	**20**	**40**	**60**	**80**	**100**
Directional Split = 50/50						
≤200	9.0	29.2	43.4	49.4	51.0	52.6
400	16.2	41.0	54.2	61.6	63.8	65.8
600	15.8	38.2	47.8	53.2	55.2	56.8
800	15.8	33.8	40.4	44.0	44.8	46.6
1,400	12.8	20.0	23.8	26.2	27.4	28.6
2,000	10.0	13.6	15.8	17.4	18.2	18.8
2,600	5.5	7.7	8.7	9.5	10.1	10.3
3,200	3.3	4.7	5.1	5.5	5.7	6.1
Directional Split = 60/40						
≤200	11.0	30.6	41.0	51.2	52.3	53.5
400	14.6	36.1	44.8	53.4	55.0	56.3
600	14.8	36.9	44.0	51.1	52.8	54.6
800	13.6	28.2	33.4	38.6	39.9	41.3
1,400	11.8	18.9	22.1	25.4	26.4	27.3
2,000	9.1	13.5	15.6	16.0	16.8	17.3
2,600	5.9	7.7	8.6	9.6	10.0	10.2
Directional Split = 70/30						
≤200	9.9	28.1	38.0	47.8	48.5	49.0
400	10.6	30.3	38.6	46.7	47.7	48.8
600	10.9	30.9	37.5	43.9	45.4	47.0
800	10.3	23.6	28.4	33.3	34.5	35.5
1,400	8.0	14.6	17.7	20.8	21.6	22.3
2,000	7.3	9.7	11.7	13.3	14.0	14.5
Directional Split = 80/20						
≤200	8.9	27.1	37.1	47.0	47.4	47.9
400	6.6	26.1	34.5	42.7	43.5	44.1
600	4.0	24.5	31.3	38.1	39.1	40.0
800	3.8	18.5	23.5	28.4	29.1	29.9
1,400	3.5	10.3	13.3	16.3	16.9	32.2
2,000	3.5	7.0	8.5	10.1	10.4	10.7
Directional Split = 90/10						
≤200	4.6	24.1	33.6	43.1	43.4	43.6
400	0.0	20.2	28.3	36.3	36.7	37.0
600	-3.1	16.8	23.5	30.1	30.6	31.1
800	-2.8	10.5	15.2	19.9	20.3	20.8
1,400	-1.2	5.5	8.3	11.0	11.5	11.9

Note: Straight-line interpolation of $f_{np,PTSF}$ for percent no-passing zones, demand flow rate, and directional split is recommended to the nearest 0.1.

Note that in Exhibit 15-21, the adjustment factor depends on the total two-way demand flow rate, even though the factor is applied to a single directional analysis. The factor reflects not only the percent of no-passing zones in the analysis segment but also the directional distribution of traffic. The directional distribution measure is the same regardless of the direction being considered. Thus, for example, splits of 70/30 and 30/70 result in the same factor, all other variables being constant. Equation 15-9, however, adjusts the factor to reflect the balance of flows in the analysis and opposing directions.

Step 7: Estimate the PFFS

This step is included only in the analysis of Class III two-lane highways. PFFS is not used in the determination of LOS for Class I or Class II facilities. The computation is straightforward, since both the FFS and the ATS have already been determined in previous steps. PFFS is estimated from Equation 15-11:

$$PFFS = \frac{ATS_d}{FFS}$$

Equation 15-11

where all terms are as previously defined.

Step 8: Determine LOS and Capacity

LOS Determination

At this point in the analysis, the values of any needed measure(s) have been determined. The LOS is found by comparing the appropriate measures with the criteria of Exhibit 15-3. The measure(s) used must be appropriate to the class of the facility being studied:

- Class I: ATS and PTSF;
- Class II: PTSF; and
- Class III: PFFS.

For Class I highways, two service measures are applied. When Exhibit 15-3 is entered, therefore, two LOS designations can be obtained. The worse of the two is the prevailing LOS. For example, if ATS results in a LOS C designation and PTSF results in a LOS D designation, LOS D is assigned.

Capacity Determination

Capacity, which exists at the boundary between LOS E and F, is not determined by a measure of effectiveness. Under base conditions, the capacity of a two-lane highway (in one direction) is 1,700 pc/h. To determine the capacity under prevailing conditions, relevant adjustment factors must be applied to Equation 15-3 and Equation 15-7. In this case, however, the demand flow rate of 1,700 pc/h under base conditions is known, and the demand flow rate under prevailing conditions is sought.

First, capacity is defined as a flow rate, so the PHF in Equation 15-3 and Equation 15-7 is set at 1.00. Then, Equation 15-12 or Equation 15-13 (or both) are applied, as described below.

Equation 15-12

$$c_{dATS} = 1{,}700 f_{g,ATS} f_{HV,ATS}$$

Equation 15-13

$$c_{dPTSF} = 1{,}700 f_{g,PTSF} f_{HV,PTSF}$$

where

c_{dATS} = capacity in the analysis direction under prevailing conditions based on ATS (pc/h), and

c_{dPTSF} = capacity in the analysis direction under prevailing conditions based on PTSF (pc/h).

For Class I highways, both capacities must be computed. The lower value represents capacity. For Class II highways, only the PTSF-based capacity is computed. For Class III highways, only the ATS-based capacity is computed.

One complication is that the adjustment factors depend on the demand flow rate (in vehicles per hour). Thus, adjustment factors for a base flow rate of 1,700 pc/h must be used. Technically, this value should be adjusted to reflect grade and heavy vehicle adjustments. This would create an iterative process in which a result is guessed and then checked.

In practical terms, this is unnecessary, since the highest flow group in all adjustment exhibits is greater than 900 veh/h. It is highly unlikely that any adjustments would reduce 1,700 pc/h to less than 900 veh/h. Therefore, in capacity determinations, all adjustment factors should be based on a flow rate greater than 900 veh/h.

Capacity may be limited by the directional distribution of traffic and the total two-way base capacity of 3,200 pc/h.

Another characteristic of this methodology must be considered in evaluating capacity. When the directional distribution is other than 50/50 (in level and rolling terrain), the two-way capacity implied by each directional capacity may be different. Moreover, the implied two-way capacity from either or both directions may be more than the limit of 3,200 pc/h. In such cases, the directional capacities estimated are not achievable with the stated directional distribution. If this is the case, then base capacity is restricted to 1,700 pc/h in the direction with the heaviest flow, and capacity in the opposing direction is found by using the opposing proportion of flow, with an upper limit of 1,500 pc/h.

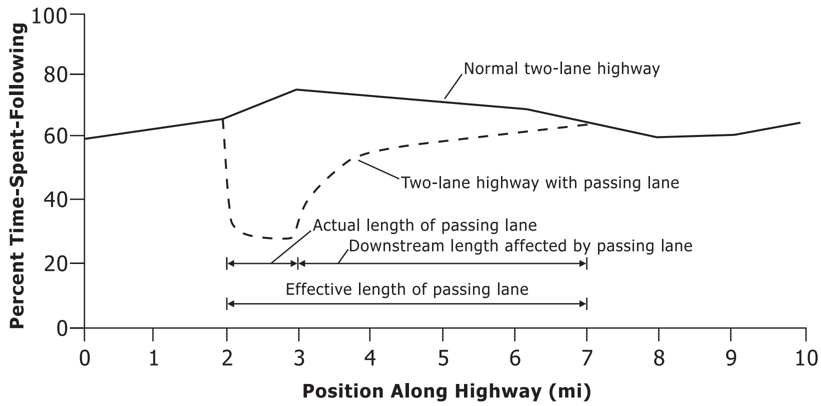
Directional Segments with Passing Lanes

Providing a passing lane on a two-lane highway in level or rolling terrain improves operational performance and therefore may improve LOS. A procedure to estimate this effect is described in this section.

This procedure should be applied only in level and rolling terrain. On specific grades, added lanes are considered to be *climbing lanes*, which are addressed in the next section.

The effective length of a passing lane is longer than its actual length.

Exhibit 15-22 illustrates the operational effect of a passing lane on PTSF. It shows that the passing lane provides operational benefits for some distance downstream before PTSF returns to its former level (without a passing lane). Thus, a passing lane's effective length is greater than its actual length.



Source: Harwood and Hoban (*6*).

Exhibit 15-22
Operational Effect of a Passing Lane on PTSF

LIVE GRAPH
Click here to view

Exhibit 15-23 gives the length of the downstream segment affected by the passing lane for both ATS and PTSF. In the case of ATS, the effect is limited to 1.7 mi in all cases. Where PTSF is concerned, however, the effect can be far longer than the passing lane itself—up to 13 mi for low demand flow rates.

Exhibit 15-23
Downstream Length of Roadway Affected by Passing Lanes on Directional Segments in Level and Rolling Terrain

Directional Demand Flow Rate, v_d (pc/h)	Downstream Length of Roadway Affected, L_{de} (mi)	
	PTSF	ATS
≤200	13.0	1.7
300	11.6	1.7
400	8.1	1.7
500	7.3	1.7
600	6.5	1.7
700	5.7	1.7
800	5.0	1.7
900	4.3	1.7
≥1,000	3.6	1.7

Note: Interpolation to the nearest 0.1 is recommended.

The analysis segment should include the entire length of the passing lane's downstream effect.

The procedure here is intended for the analysis of directional segments in level or rolling terrain that encompass the entire passing lane. Segments of the highway upstream and downstream of the passing lane may be included in the analysis. It is recommended that the analysis segment include the full length of the passing lane's downstream effect.

Care should be taken in considering the effect of passing lanes on service flow rates; they are greatly affected by the length of the passing lane relative to the length of the analysis segment.

Because of the downstream effect on PTSF, the LOS on a two-lane highway segment that is determined by PTSF (Class I and Class II) may be significantly improved by the addition of a passing lane. Care must be taken, however, in considering the impact of a passing lane on service volumes or service flow rates. The result is highly dependent on the relative lengths of the analysis segment and the passing lane. If the analysis segment includes only the length of the passing lane and its downstream effective length (on PTSF), the passing lane may appear to increase service flow rates dramatically at LOS A–D (capacity, and therefore LOS E, would not be affected). However, if additional lengths are included in the analysis segment, this impact is reduced, sometimes considerably. Thus, apparent increases in service volumes or service flow rates must be carefully considered in the context of how they were obtained.

The steps in this special analysis procedure are as follows.

Step 1: Conduct an Analysis Without the Passing Lane

The first step in the operational analysis of the impact of a passing lane is to conduct the basic analysis steps described previously. The remainder of the procedure essentially predicts the improvement caused by the passing lane compared with a similar segment without a passing lane.

Step 2: Divide the Segment into Regions

The analysis segment can be divided into four regions, as follows:

1. Length upstream of the passing lane L_u,
2. Length of the passing lane L_{pl},
3. Length downstream of the passing lane within its effective length L_{de}, and
4. Length downstream of the passing lane beyond its effective length L_d.

Some of these regions may not be involved in a particular analysis. Region 2, the passing lane, must be included in every analysis. In addition, it is strongly recommended, but not absolutely necessary, that Region 3 be included. Regions 1 and 4 are optional, and inclusion is at the discretion of the analyst.

The four lengths must add up to the total length of the analysis segment. The analysis regions and their lengths will differ for estimations of ATS and PTSF, as the downstream effects indicated in Exhibit 15-23 differ for each.

The length of the passing lane L_{pl} is either the length of the passing lane as constructed or the planned length. It should include the length of the lane addition as well as the length of the entrance and exit tapers. The procedure is calibrated for passing lanes within the optimal lengths shown in Exhibit 15-24. Passing lanes that are substantially shorter or longer than the optimums shown may provide less operational benefit than predicted by this procedure.

Exhibit 15-24
Optimal Lengths of Passing Lanes on Two-Lane Highways

Directional Demand Flow Rate, v_d (pc/h)	Optimal Passing Lane Length (mi)
≤100	≤0.50
>100 ≤400	>0.50 ≤0.75
>400 ≤700	>0.75 ≤1.00
≥700	>1.00 ≤2.00

The length of the conventional two-lane highway segment upstream of the passing lane L_u is determined by the actual or planned placement of the passing lane within the analysis segment. The length of the downstream highway segment within the effective length of the passing lane L_{de} is determined from Exhibit 15-23. Any remaining length of the analysis segment downstream of the passing lane is included in L_d, which is computed from Equation 15-14:

Equation 15-14

$$L_d = L_t - \left(L_u + L_{pl} + L_{de}\right)$$

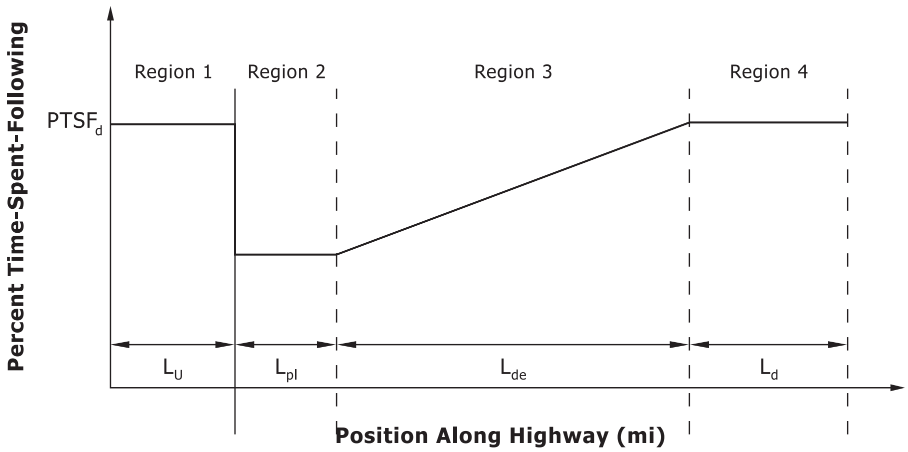
where L_t is the total length of the analysis segment in miles and all other terms are as previously defined.

Step 3: Determine the PTSF

PTSF within lengths L_u and L_d is assumed to be equal to the $PTSF_d$ as predicted by the normal analysis procedure (without a passing lane). Within the segment with the passing lane L_{pl}, PTSF is generally equal to 58% to 62% of its

upstream value. This effect is a function of the directional demand flow rate. Within L_{de}, the PTSF is assumed to increase linearly from the passing lane value to the normal upstream value. This distribution is illustrated in Exhibit 15-25.

Exhibit 15-25
Effect of a Passing Lane on PTSF



On the basis of this model, the PTSF for the entire analysis segment, as affected by the passing lane, is given by Equation 15-15:

Equation 15-15

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + L_d + f_{pl,PTSF} L_{pl} + \left(\frac{1 + f_{pl,PTSF}}{2} \right) L_{de} \right]}{L_t}$$

where

$PTSF_{pl}$ = percent time-spent-following for segment as affected by the presence of a passing lane (decimal); and

$f_{pl,PTSF}$ = adjustment factor for the impact of a passing lane on percent time-spent-following, from Exhibit 15-26.

All other variables are as previously defined.

Exhibit 15-26
Adjustment Factor for the Impact of a Passing Lane on PTSF ($f_{pl,PTSF}$)

Directional Demand Flow Rate, v_d (pc/h)	$f_{pl,PTSF}$
≤100	0.58
200	0.59
300	0.60
400	0.61
500	0.61
600	0.61
700	0.62
800	0.62
≥900	0.62

Note: Interpolation is not recommended; use closest value.

If the analysis segment cannot encompass the entire length L_{de} because it is truncated by a town or major intersection within it, then distance L_d is not used. Therefore, the actual downstream length within the analysis segment L'_{de} is less than the value of L_{de} tabulated in Exhibit 15-23. In this case, Equation 15-16 should be used instead of Equation 15-15:

Equation 15-16

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + f_{pl,PTSF} L_{pl} + f_{pl,PTSF} L'_{de} + \left(\frac{1 - f_{pl,PTSF}}{2} \right) \left(\frac{L'^{2}_{de}}{L_{de}} \right) \right]}{L_t}$$

where all terms are as previously defined.

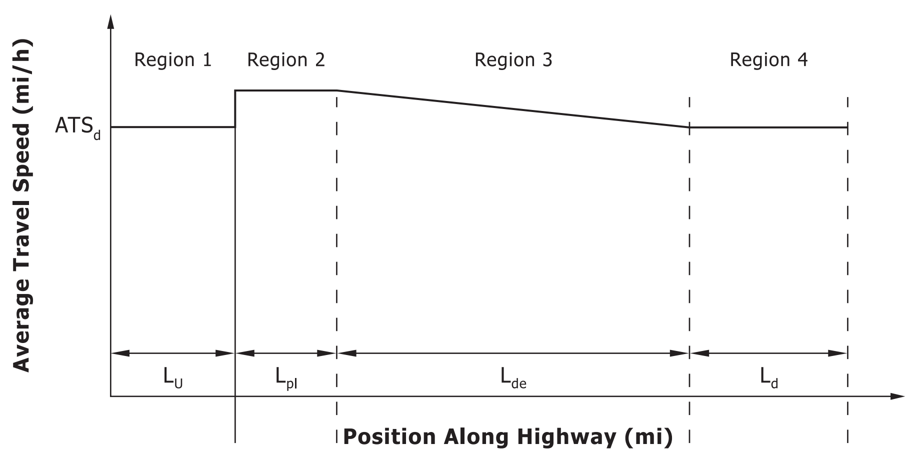
In general, the effective downstream distance of the passing lane should not be truncated. A downstream boundary short of the effective downstream distance should be considered at the point where any of the following occur:

- The environment of the highway radically changes, as in the case of entering a small town or developed area from a rural segment;
- A major unsignalized intersection is present, leading to a change in the demand flow rate;
- A proximate signalized intersection begins to affect the operation of the two-lane segment;
- The terrain changes significantly; and
- Lane or shoulder widths change significantly.

Step 4: Determine the ATS

The ATS within lengths L_u and L_d is assumed to be equal to ATS_d, the speed that would exist without the passing lane. Within the passing lane, the ATS is generally between 8% and 11% higher than its upstream value, depending on the directional demand flow rate. Within the effective downstream length, L_{de}, ATS is assumed to decrease linearly with the distance from the passing lane, from the passing lane value to the normal value. Exhibit 15-27 illustrates the impact of a passing lane on ATS.

Exhibit 15-27
Impact of a Passing Lane on ATS



The ATS is computed with Equation 15-17:

Equation 15-17

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + L_d + \left(\frac{L_{pl}}{f_{pl,ATS}} \right) + \left(\frac{2L_{de}}{1 + f_{pl,ATS}} \right)}$$

where

ATS_{pl} = average travel speed in the analysis segment as affected by a passing lane (mi/h); and

$f_{pl,ATS}$ = adjustment factor for the effect of a passing lane on ATS, from Exhibit 15-28.

All other variables are as previously defined.

Exhibit 15-28
Adjustment Factor for Estimating the Impact of a Passing Lane on ATS ($f_{pl,ATS}$)

Directional Demand Flow Rate, v_d (pc/h)	$f_{pl,ATS}$
≤100	1.08
200	1.09
300	1.10
400	1.10
500	1.10
600	1.11
700	1.11
800	1.11
≥900	1.11

Note: Interpolation is not recommended; use closest value.

In the case where the analysis segment cannot include all of the effective downstream distance, L_{de}, because a town or major intersections cause the segment to be truncated, distance L'_{de} is less than the value of L_{de}. In this case, Equation 15-18 is used instead of Equation 15-17 to compute ATS.

Equation 15-18

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + \dfrac{L_{pl}}{f_{pl,ATS}} + \dfrac{2L'_{de}}{\left[1 + f_{pl,ATS} + \left(f_{pl,ATS} - 1\right)\left(\dfrac{L_{de} - L'_{de}}{L_{de}}\right)\right]}}$$

where all terms are as previously defined.

Step 5: Determine the LOS

Determining the LOS for a segment with a passing lane is no different from determining the LOS for a normal segment, except that ATS_{pl} and $PTSF_{pl}$ are used as the service measures with the criteria of Exhibit 15-3.

As with a normal segment, LOS for Class I highways is based on both PTSF and ATS. LOS for Class II highways is based only on PTSF. Class III highways would not normally have passing lanes, but if such a situation arose, PFFS = ATS/FFS would be used to determine LOS.

Directional Segments with Climbing Lanes on Upgrades

A climbing lane is, in effect, a passing lane added on an upgrade to allow traffic to pass heavy vehicles whose speeds are reduced. Generally, a lane is added to the right, and all slow-moving vehicles should move to this lane, allowing faster vehicles to pass in the normal lane.

The American Association of State Highway and Transportation Officials (*7*) indicates that climbing lanes on two-lane highways are warranted when

- The directional flow rate on the upgrade exceeds 200 veh/h;
- The directional flow rate for trucks on the upgrade exceeds 20 veh/h; and

- Any of the following conditions apply:
  - A speed reduction of 10 mi/h or more exists for a typical truck;
  - LOS E or F exists on the upgrade without a climbing lane; or
  - Without a climbing lane, the LOS is two or more levels lower on the upgrade than on the approach segment to the grade.

An operational analysis of the impact of a climbing lane on a two-lane highway is performed with the same procedures as passing lanes in level or rolling terrain, with three major differences:

1. Adjustment factors for the existence of the climbing lane are taken from Exhibit 15-29,
2. The analysis without a climbing lane is conducted by using the specific grade procedures, and
3. Distances L_u and L_d are set to zero.

The effective downstream distance L_{de} is also generally set to zero unless the climbing lane ends before the grade does. In this case, a value less than the values typically used should be considered.

Exhibit 15-29
Adjustment Factors (f_{pl}) for Estimating ATS and PTSF Within a Climbing Lane

Directional Demand Flow Rate, v_d (pc/h)	f_{pl} ATS	f_{pl} PTSF
0–300	1.02	0.20
>300–600	1.07	0.21
>600	1.14	0.23

LOS Assessment for Directional Two-Lane Facilities

Two-lane highway segments have uniform characteristics that provide a basis for their analysis. Several contiguous two-lane highway segments (in the same directions) may be combined to look at a longer section (with varying characteristics) as a facility. A separate operational analysis would have to be done for each uniform segment within the facility.

Weighted-average values of PTSF and ATS may be estimated for the facility. The weighting is on the basis of total travel time within the 15-min analysis period. The total travel time of all vehicles within the 15-min analysis period is estimated with Equation 15-19 and Equation 15-20:

Equation 15-19

$$VMT_{i15} = 0.25\left(\frac{V_i}{PHF}\right)L_t$$

Equation 15-20

$$TT_{i15} = \frac{VMT_{i15}}{ATS_i}$$

where

VMT_{i15} = total vehicle miles traveled by all vehicles in directional segment i during the 15-min analysis period (veh-mi),

V_i = demand volume in directional segment i (veh/h),

PHF = peak hour factor,

L_t = total length of directional segment i (mi),

TT_{i15} = total travel time consumed by all vehicles traversing directional segment i during the 15-min analysis period (veh-h), and

ATS_i = average travel speed for directional segment i (mi/h).

Once the total travel time for all vehicles in each segment is computed, weighted-average values of PTSF and ATS can be obtained with Equation 15-21 and Equation 15-22:

$$ATS_F = \frac{VMT_1 + VMT_2 + VMT_3 + ... + VMT_i}{TT_1 + TT_2 + TT_3 + ... + TT_i}$$

Equation 15-21

$$PTSF_F = \frac{(TT_1 \times PTSF_1) + (TT_2 \times PTSF_2) + (TT_3 \times PTSF_3) + ... + (TT_i \times PTSF_i)}{TT_1 + TT_2 + TT_3 + ... + TT_i}$$

Equation 15-22

where

ATS_F = average travel speed for the facility (mi/h),

$PTSF_F$ = percent time-spent-following for the facility (decimal),

$PTSF_i$ = percent time-spent-following for segment i (decimal),

VMT_i = vehicle miles traveled for segment i (veh-mi), and

TT_i = total travel time of all vehicles in segment i (veh-h).

When a facility is put together, two-lane highway segments of different classes should not be combined. Levels of service for the facility are still based on the criteria of Exhibit 15-3. Class III two-lane highways generally only exist in short segments and would not be expected to cover a distance long enough to form a facility.

Other Performance Measures

This chapter provides detailed methodologies for estimating three measures of effectiveness that are used (depending on the highway class) to determine LOS:

- ATS (mi/h, Class I and Class III highways),
- PTSF (decimal, Class I and Class II highways), and
- PFFS (decimal, Class III highways).

In the previous section, two additional measures were introduced that can be considered as performance measures, even though they are not used to determine LOS. Equation 15-19 and Equation 15-20 can be used to estimate

- Total vehicle miles traveled by all vehicles in the analysis segment during the 15-min analysis period VMT_{i15} (veh-mi), and
- Total travel time consumed by all vehicles traversing the analysis segment during the 15-min analysis period TT_{i15} (veh-h).

These values may also be of interest in fully understanding the operational quality of the study segment.

A volume-to-capacity (v/c) ratio is also a common performance measure of interest in LOS and capacity analysis. It is most easily computed for two-lane highways with Equation 15-23:

Equation 15-23

$$v/c = \frac{v_d}{1{,}700}$$

where v_d is the directional demand flow rate, converted to equivalent base conditions.

The difficulty in this is that there may be two values of v_d: one for estimating ATS and another for estimating PTSF (depending on the class of highway). For Class I highways, where both measures are used, the result yielding the highest *v/c* ratio would be used. For Class II highways, only PTSF is used, and only one value would exist. For Class III highways, only ATS is used, and only one value would exist.

BICYCLE MODE

The calculation of bicycle LOS on multilane and two-lane highways shares the same methodology, since multilane and two-lane highways operate in fundamentally the same manner for bicyclists. Cyclists travel much more slowly than the prevailing traffic flow, staying as far to the right as possible and using paved shoulders when available, indicating the need for only one model.

The bicycle LOS model for two-lane and multilane highways uses a traveler-perception model calibrated by using a linear regression (*4*). The model fits independent variables associated with roadway characteristics to the results of a user survey that rated the comfort of various bicycle facilities. The resulting bicycle LOS score generally ranges from 0.5 to 6.5 and is stratified to produce a LOS A–F result, on the basis of Exhibit 15-4.

Step 1: Gather Input Data

The methodology requires gathering the following input data for the facility in question:

1. Lane width (ft),
2. Shoulder width (ft),
3. Hourly directional motorized vehicle volume (veh/h),
4. Number of directional through lanes (needed for multilane highways),
5. Percentage of heavy vehicles (decimal),
6. Posted speed limit (mi/h),
7. Percentage of segment with occupied on-highway parking (decimal), and
8. Pavement rating.

Pavement rating is determined by using FHWA's 5-point present serviceability rating scale (*8*): 1 (very poor), 2 (poor), 3 (fair), 4 (good), and 5 (very good). Where data for specific variables are not available, default values may be used as shown in Exhibit 15-5.

Step 2: Calculate the Directional Flow Rate in the Outside Lane

On the basis of the hourly directional volume, the peak hour factor, and the number of directional lanes (one for basic two-lane highways, two or more for

passing lanes or multilane highways), calculate the directional demand flow rate of motorized traffic in the outside lane with Equation 15-24:

$$v_{OL} = \frac{V}{PHF \times N}$$

Equation 15-24

where

v_{OL} = directional demand flow rate in the outside lane (veh/h),

V = hourly directional volume (veh/h),

PHF = peak hour factor, and

N = number of directional lanes (=1 for two-lane highways).

Step 3: Calculate the Effective Width

The effective width of the outside through lane depends on both the actual width of the outside through lane and the shoulder width, since cyclists will be able to travel in the shoulder where one is provided. Moreover, striped shoulders of 4 ft or greater provide more security to cyclists by giving cyclists a dedicated place to ride outside of the motorized vehicle travelway. Thus, an 11-ft lane and adjacent 5-ft paved shoulder results in a larger effective width for cyclists than a 16-ft lane with no adjacent shoulder.

Parking occasionally exists along two-lane highways, particularly in developed areas (Class III highways) and near entrances to recreational areas (Class II and Class III highways) where a fee is charged for off-highway parking or where the off-highway parking is inadequate for the parking demand. On-highway parking reduces the effective width, because parked vehicles take up shoulder space and bicyclists leave some shy distance between themselves and the parked cars.

Equation 15-25 through Equation 15-29 are used to calculate the effective width, W_e, on the basis of the paved shoulder width, W_s, and the hourly directional volume, V:

If W_s is greater than or equal to 8 ft:

$$W_e = W_v + W_s - (\%OHP \times 10\text{ ft})$$

Equation 15-25

If W_s is greater than or equal to 4 ft and less than 8 ft:

$$W_e = W_v + W_s - 2 \times (\%OHP(2\text{ ft} + W_s))$$

Equation 15-26

If W_s is less than 4 ft:

$$W_e = W_v + (\%OHP(2\text{ ft} + W_s))$$

Equation 15-27

with, if V is greater than 160 veh/h:

$$W_v = W_{OL} + W_s$$

Equation 15-28

Otherwise,

$$W_v = (W_{OL} + W_s) \times (2 - 0.005V)$$

Equation 15-29

where

W_v = effective width as a function of traffic volume (ft),

W_{OL} = outside lane width (ft),

W_s = paved shoulder width (ft),

V = hourly directional volume (veh/h),

W_e = average effective width of the outside through lane (ft), and

$\%OHP$ = percentage of segment with occupied on-highway parking (decimal).

Step 4: Calculate the Effective Speed Factor

The effect of motor vehicle speed on bicycle quality of service is primarily related to the differential between motor vehicle and bicycle travel speeds. For instance, a typical cyclist may travel in the range of 15 mi/h. An increase in motor vehicle speeds from 20 to 25 mi/h is more readily perceived than a speed increase from 60 to 65 mi/h, since the speed differential increases by 100% in the first instance compared with only 11% in the latter. Equation 15-30 shows the calculation of the effective speed factor that accounts for this diminishing effect.

Equation 15-30

$$S_t = 1.1199\ln(S_p - 20) + 0.8103$$

where

S_t = effective speed factor, and

S_p = posted speed limit (mi/h).

Step 5: Determine the LOS

With the results of Steps 1–4, the bicycle LOS score can be calculated from Equation 15-31:

Equation 15-31

$$BLOS = 0.507\ln(v_{OL}) + 0.1999S_t(1 + 10.38HV)^2 + 7.066(1/P)^2 - 0.005(W_e)^2 + 0.057$$

where

$BLOS$ = bicycle level of service score;

v_{OL} = directional demand flow rate in the outside lane (veh/h);

HV = percentage of heavy vehicles (decimal); if $V < 200$ veh/h, then HV should be limited to a maximum of 50%;

P = FHWA's 5-point pavement surface condition rating; and

W_e = average effective width of the outside through lane (ft).

Finally, the BLOS score value is used in Exhibit 15-4 to determine the bicycle LOS for the segment.

3. APPLICATIONS

This chapter provides methodologies for the analysis of two-lane highway uninterrupted-flow segments that serve a wide variety of travel purposes. The procedures are most easily applied in the operational analysis mode to determine the capacity and LOS of a two-lane highway segment with known characteristics. Other applications are also possible.

DEFAULT VALUES

A detailed report on the use of default values in uninterrupted-flow analysis, including the analysis of two-lane highways, is given elsewhere (*4*). Specific default values for use with the methodology of this chapter were given in Exhibit 15-5. Default values may also be based on local estimates developed from past observations of a specific site or similar sites in a given jurisdiction.

For operational analysis and design analysis, the use of default values should be minimized whenever possible. Every default value used to replace a field-measured or other site-specific value introduces additional uncertainty into the estimation process and into the accuracy of results. Nevertheless, where no site-specific values are available, default values allow at least an approximate analysis of the situation. For planning and preliminary design analysis, use of default values is generally required, since few details are available at this stage of consideration.

TYPES OF ANALYSIS

Operational Analysis

All geometric, development, and traffic-demand characteristics are provided. The LOS that is expected to exist during the analysis period is estimated. A number of alternative performance measures may also be estimated. The methodology of this chapter is most easily used in this mode.

Design Analysis

In design analysis, demand characteristics are generally known. The analysis is intended to give insights into design parameters needed to provide a target LOS for the demand characteristics as stated. For two-lane highways, design decisions are relatively limited. Lane and shoulder widths have a moderate impact on operations but generally do not result in a markedly different LOS.

Typical design projects include horizontal or vertical curve realignments, which may affect percent no-passing zones and free-flow speeds.

The special procedures outlined in this chapter to consider the impacts of passing lanes and climbing lanes can be used to provide critical design insight. However, the computations are performed in the operational analysis mode, leading to a comparison of operations with or without the passing or climbing lane.

This chapter's appendix deals with some special design issues related to two-lane highways. However, there is no methodology at this point for estimating the impact of these design treatments on operating quality.

Given the relatively few design parameters involved in a two-lane highway, most design analysis is conducted as an iterative series of operational analyses.

Planning and Preliminary Engineering Analysis

Planning and preliminary engineering analysis has the same objectives as design analysis, except that it occurs early in the process when few details of demand and other characteristics are known. Thus, design analysis is augmented by the use of default values for many inputs.

The other principal characteristic of planning and preliminary engineering analysis is that demands are generally described in terms of two-way AADT.

This chapter includes generalized daily service volume tables covering a specific range of default values. They can be used for a coarse and general evaluation of the likely LOS for a two-lane highway in various settings under an expected AADT demand. These tables are useful only for the most preliminary of analyses. For example, all two-lane highway segments in a particular region can be considered by using these criteria. Any segments that appear to be operating at an undesirable LOS should be subjected to site-specific study with a more detailed operational analysis before any major design, reconstruction, or investment decisions are made.

SERVICE FLOW RATES, SERVICE VOLUMES, AND DAILY SERVICE VOLUMES

Service flow rates, service volumes, and daily service volumes are useful concepts that can be used in the analysis of many types of facilities, including two-lane highways. The three terms must be clearly understood, because they are very different.

1. Service flow rates SF_i represent the maximum directional rate of flow that can be accommodated by a segment while maintaining the designated LOS *i*.
2. Service volumes SV_i represent the maximum directional hourly volume that can be accommodated by a segment while maintaining the designated LOS *i* during the worst 15-min period of the hour.
3. Daily service volumes DSV_i represent the maximum AADT that can be accommodated by a segment while maintaining the designated LOS *i* during the worst 15 min of the peak hour of the day, in the highest-flow direction.

In general, service flow rates and service volumes are directional values, while the daily service volume is usually stated as total traffic in both directions (since that is how AADT is stated).

The service flow rate for a particular LOS is estimated by using the methodology for the segment type under study (two-lane highways in this

chapter). Equation 15-32 is then used to estimate service volume for a segment, and Equation 15-33 is used to estimate daily service volume for a segment.

$$SV_i = SF_i \times PHF$$

Equation 15-32

$$DSV_i = \frac{SV_i}{K \times D}$$

Equation 15-33

where K is the proportion of traffic occurring in the peak hour for the study segment and D is the proportion of traffic occurring in the peak direction for the study segment.

For two-lane highways, several complications arise. While all analyses of two-lane highways are for one direction, the two directions interact. Thus, if a two-way daily service volume is estimated by using the service flow rate in one direction, and then again in the other direction, different results could easily be obtained.

As with all uninterrupted-flow segments, capacity is synonymous with the service flow rate for LOS E. Thus, Equation 15-12 and Equation 15-13, presented earlier, may be used to estimate service flow rates for LOS E. Even in this case, there are two equations, since the value will depend on whether ATS or PTSF is the determining LOS parameter.

For other levels of service, the process of determining a service flow rate is more complicated. It would be beneficial if the methodology of this chapter could be used in reverse—that is, start with a value of ATS or PTSF and work backwards to the demand flow rate that would create that value. Unfortunately, virtually all of the adjustment factors used in this process depend on the demand flow rate, which is what the analyst would be trying to find. Such computations would therefore be iterative. Finding appropriate service flow rates for each LOS requires an iterative process in which different flow rates are incrementally used until the threshold for a particular LOS is found.

Once service flow rates are found, Equation 15-32 and Equation 15-33 can be used to infer service volumes and daily service volumes.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 15-30 shows generalized daily service volumes for use in planning and preliminary design. The exhibit provides daily service volume values for three types of segments: (*a*) a Class I highway in level terrain, (*b*) a Class I highway in rolling terrain, and (*c*) a Class II highway in rolling terrain.

Typical conditions assumed for each are given below the table. Various values of *K*- and *D*-factors are given. Since these values vary greatly from region to region, the analyst must select the values most appropriate to the particular application. Interpolation may be used, if desired, to obtain intermediate values.

Exhibit 15-30
Generalized Daily Service Volumes for Two-Lane Highways

The Class I—level example assumes higher speeds, with significant passing opportunities.

The Class I—rolling example assumes more moderate speeds and reduced passing opportunities because of the terrain.

The Class II—rolling example is similar to a scenic or recreational highway with lower speeds and limited passing opportunities.

K-Factor	*D*-Factor	Class I—Level				Class I—Rolling				Class II—Rolling			
		LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
0.09	50%	5.5	9.3	16.5	31.2	4.2	8.4	15.7	30.3	5.0	9.8	18.2	31.2
	55%	4.9	8.7	14.9	30.2	3.7	7.9	14.0	29.2	4.1	8.7	16.0	30.2
	60%	4.4	8.1	13.9	27.6	3.7	6.2	12.8	26.8	3.7	7.9	14.6	27.6
	65%	4.1	7.9	12.9	25.5	3.4	5.9	11.4	24.7	3.3	5.9	13.2	25.5
0.10	50%	5.0	8.4	14.8	28.0	3.8	7.6	14.2	27.2	4.4	8.8	16.3	28.0
	55%	4.4	7.9	13.4	27.1	3.3	7.1	12.6	26.3	3.7	7.9	14.4	27.1
	60%	4.0	7.3	12.5	24.9	3.3	5.6	11.5	24.1	3.3	7.1	13.1	24.9
	65%	3.7	7.1	11.6	23.0	3.0	5.3	10.3	22.3	3.0	5.3	11.9	23.0
0.12	50%	4.1	7.0	12.4	23.4	3.1	6.3	11.8	22.7	3.7	7.4	13.6	23.4
	55%	3.7	6.5	11.2	22.6	2.8	5.9	10.5	21.9	3.1	6.5	12.0	22.6
	60%	3.3	6.1	10.4	20.7	2.7	4.7	9.6	20.1	2.7	5.9	10.9	20.7
	65%	3.1	5.9	9.6	19.1	2.5	4.4	8.5	18.5	2.4	4.4	9.9	19.1
0.14	50%	3.5	6.0	10.6	20.0	2.7	5.4	10.1	19.4	3.2	6.3	11.7	20.0
	55%	3.1	5.6	9.6	19.4	2.4	5.1	9.0	18.8	2.6	5.6	10.3	19.4
	60%	2.8	5.2	8.9	17.7	2.3	4.0	8.2	17.2	2.3	5.1	9.4	17.7
	65%	2.6	5.1	8.2	16.4	2.1	3.8	7.3	15.9	2.1	3.8	8.5	16.4

Notes: Volumes are thousands of vehicles per day.
Assumed values for all entries: 10% trucks, PHF = 0.88, 12-ft lanes, 6-ft shoulders, 10 access points/mi.
Assumed values for Class I—level: BFFS = 65 mi/h, 20% no-passing zones.
Assumed values for Class I—rolling: BFFS = 60 mi/h, 40% no-passing zones.
Assumed values for Class II—rolling: BFFS = 50 mi/h, 60% no-passing zones.

A number of interesting characteristics are displayed in Exhibit 15-30:

1. LOS A is not shown. Even in level terrain, it is possible to achieve this level only at very low demand flow rates (almost always lower than 50 veh/h, directional).
2. The range of demand flows falling within LOS E is broad compared with other levels of service. This is because the quality of service on two-lane highways tends to become unacceptable at relatively low v/c ratios. Few two-lane highways are observed operating at or near capacity (except for short segments), because most will have been expanded before capacity demand flows develop.

Exhibit 15-30 should be used only in generalized planning and preliminary engineering analysis. It is best used to examine a number of two-lane highways within a given jurisdiction to determine which need closer scrutiny. If anticipated AADTs on a given segment or facility appear to put the segment or facility into an undesirable LOS, then more site-specific data should be obtained (or forecast) and a full operational analysis conducted before any firm commitments to reconstruct or improve the highway are made.

USE OF ALTERNATIVE TOOLS

No alternative deterministic tools are in common use for two-lane highway analysis. Two-lane highway simulation tools are in various stages of development, but user experience with these tools is insufficient to support the formulation of useful guidance for their application to extend the scope of the procedures described in this chapter.

One of the potentially useful features of two-lane highway simulation is the ability to model specific configurations of a series of no-passing zones, exclusive passing lanes, and access points, all of which are now described in general terms (e.g., percent no-passing zones) in this chapter. Network simulation tools can also include traffic control devices at specific points.

It is possible to obtain additional performance measures from simulation results. One example is *follower density,* which is defined in terms of the number of followers per mile per lane. This concept, which is discussed in more detail in Chapter 24, Concepts: Supplemental, has attracted increasing international interest. Some examples that illustrate potential uses of two-lane highway simulation are presented elsewhere (*9*).

4. EXAMPLE PROBLEMS

Exhibit 15-31
List of Example Problems

Problem Number	Description	Type of Analysis
1	Find the LOS of a Class I highway in rolling terrain	Operational analysis
2	Find the LOS of a Class II highway in rolling terrain	Operational analysis
3	Find the LOS of a Class III highway in level terrain	Operational analysis
4	Find the LOS of a Class I highway with a passing lane	Operational analysis
5	Find the future bicycle LOS of a two-lane highway	Planning analysis

EXAMPLE PROBLEM 1: CLASS I HIGHWAY LOS

The Facts

A segment of Class I two-lane highway has the following known characteristics:

- Demand volume = 1,600 pc/h (total in both directions)
- Directional split (during analysis period) = 50/50
- PHF = 0.95
- 50% no-passing zones in the analysis segment (both directions)
- Rolling terrain
- 14% trucks; 4% RVs
- 11-ft lane widths
- 4-ft usable shoulders
- 20 access points/mi
- 60-mi/h BFFS
- 10-mi segment length

Find the expected LOS in each direction on the two-lane highway segment as described.

Comments

The problem statement calls for finding the LOS in each direction on a segment in rolling terrain. Because the directional split is 50/50, the solution in one direction will be the same as the solution in the other direction, so only one operational analysis needs to be conducted. The result will apply equally to each direction.

Because this is a Class I highway, both ATS and PTSF must be estimated to determine the expected LOS.

Step 1: Input Data

All input data were specified above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2 and adjustment factors found in Exhibit 15-7 (for lane and shoulder width) and Exhibit 15-8 (for access points in both directions). For 11-ft lane widths and 4-ft usable shoulders, the adjustment factor

for these features f_{LS} is 1.7 mi/h; for 20 access points/mi, the adjustment factor f_A is 5.0 mi/h. Then

$$FFS = BFFS - f_{LS} - f_A$$

$$FFS = 60.0 - 1.7 - 5.0 = 53.3 \text{ mi/h}$$

Step 3: Demand Adjustment for ATS

The demand volume must be adjusted to a flow rate in passenger cars per hour under equivalent base conditions. This is accomplished with Equation 15-3:

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

Since the demand split is 50/50, both the analysis direction and opposing demand volumes are 1,600/2 = 800 veh/h.

The grade adjustment factor $f_{g,ATS}$ is selected from Exhibit 15-9 for rolling terrain. The table is entered with a demand flow rate v_{vph} in vehicles per hour, or 800/0.95 = 842 veh/h. By interpolation in Exhibit 15-9 between 800 and 900 veh/h, the factor is 0.99 to the nearest 0.01.

The passenger car equivalent for trucks and RVs is obtained from Exhibit 15-11, again for a demand flow rate of 842 veh/h. Again, by interpolation between 800 and 900 veh/h, the values obtained are E_T = 1.4 and E_R = 1.1. The heavy vehicle adjustment is then computed with Equation 15-4:

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV,ATS} = \frac{1}{1 + 0.14(1.4 - 1) + 0.04(1.1 - 1)}$$

$$f_{HV,ATS} = 0.943$$

Then

$$v_{d,ATS} = v_{o,ATS} = \frac{800}{0.95 \times 0.99 \times 0.943} = 902 \text{ pc/h}$$

Step 4: Estimate ATS

The ATS is estimated with Equation 15-6. The adjustment factor $f_{np,ATS}$ is found in Exhibit 15-15 for an FFS of 53.3 mi/h, 50% no-passing zones, and an opposing demand flow of 902 veh/h. This selection must use interpolation on all three scales. Note that interpolation is only to the nearest 0.1 for this adjustment factor. Exhibit 15-32 illustrates the interpolation.

Exhibit 15-32
Interpolation for ATS Adjustment Factor

v_o (veh/h)	Factor for FFS = 55 mi/h 40% NPZ	50% NPZ	60% NPZ	Factor for FFS = 50 mi/h 40% NPZ	50% NPZ	60% NPZ
800	0.7	*0.9*	1.1	0.6	*0.75*	0.9
902		**0.8**			**0.65**	
1,000	0.6	*0.7*	0.8	0.4	*0.55*	0.7

Notes: $f_{np,ATS}$ = 0.65 + (0.8 - 0.65) (3.3 / 5.0) = 0.749 = **0.7**.
NPZ = no-passing zones.

Then, Equation 15-6 gives the following:

$$ATS = FFS - 0.00776(v_d + v_o) - f_{np,ATS}$$

$$ATS = 53.3 - 0.00776(902 + 902) - 0.7$$

$$ATS = 53.3 - 14.0 - 0.7 = 38.6 \text{ mi/h}$$

Step 5: Demand Adjustment for PTSF

The adjusted demand used to estimate PTSF is found with Equation 15-7 and Equation 15-8. The grade adjustment factor is taken from Exhibit 15-16 for rolling terrain and a demand flow rate of 800/0.95 = 842 pc/h. Passenger car equivalents for trucks and RVs are taken from Exhibit 15-18. In both exhibits, the demand flow rate of 842 pc/h is interpolated between 800 pc/h and 900 pc/h to obtain the correct values. The following values are obtained:

$f_{g,PTSF}$ = 1.00

E_T = 1.0

E_R = 1.0

Then, use of Equation 15-8 gives the following:

$$f_{HV,PTSF} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV,PTSF} = \frac{1}{1 + 0.14(1.0 - 1) + 0.04(1.0 - 1)} = 1.00$$

Equation 15-7 gives

$$v_{i,PTSF} = \frac{V_i}{PHF \times f_{g,PTSF} \times f_{HV.PTSF}}$$

$$v_{d,PTSF} = v_{o,PTSF} = \frac{800}{0.95 \times 1.00 \times 1.00} = 842 \text{ pc/h}$$

Step 6: Estimate PTSF

PTSF is estimated with Equation 15-9 and Equation 15-10. Exhibit 15-20 is used to obtain exponents *a* and *b* for Equation 15-10, and Exhibit 15-21 is used to obtain the no-passing-zone adjustment for Equation 15-9. All three require interpolation.

Exponents *a* and *b* are based on the opposing flow rate of 842 pc/h, which is interpolated between tabulated values of 800 pc/h and 1,000 pc/h. This is illustrated in Exhibit 15-33.

Exhibit 15-33
Interpolation for Exponents *a* and *b* for Equation 15-10

Opposing Flow Rate (pc/h)	*a*	*b*
800	-0.0045	0.833
842	**-0.0046**	**0.832**
1,000	-0.0049	0.829

Then, use of Equation 15-10 gives

$$BPTSF = 100\left[1 - \exp\left(av_d^b\right)\right]$$

$$BPTSF = 100\left[1 - \exp\left(-0.0046 \times 842^{0.832}\right)\right]$$

$$BPTSF = 71.3\%$$

The adjustment factor for no-passing zones must also be interpolated in two variables. Exhibit 15-21 is entered with 50% no-passing zones, a 50/50 directional split of traffic, and a total two-way demand flow rate of 842 + 842 = 1,684 pc/h. The interpolation is illustrated in Exhibit 15-34.

Exhibit 15-34
Interpolation for $f_{np,PTSF}$ for Equation 15-9

Total Flow Rate (pc/h)	Adjustment Factor for 40% NPZ	Adjustment Factor for 50% NPZ	Adjustment Factor for 60% NPZ
1,400	23.8	*25.0*	26.2
1,684		16.6 + (25.0 - 16.6) (316 / 600) = **21.0**	
2,000	15.8	*16.6*	17.4

Note: NPZ = no-passing zones.

Then, use of Equation 15-9 gives

$$PTSF = BPTSF + f_{np,PTSF}\left(\frac{v_{d,PTSF}}{v_{d,PTSF} + v_{o,PTSF}}\right)$$

$$PTSF = 71.3 + 21.0\left(\frac{842}{842 + 842}\right) = 81.8\%$$

Step 7: Estimate PFFS

This step is only used for Class III highways.

Step 8: Determine LOS and Capacity

LOS is determined by comparing the estimated values of ATS and PTSF with the criteria of Exhibit 15-3. An ATS of 38.6 mi/h suggests that LOS E will exist. A PTSF of 81.8% suggests that LOS E will exist. Thus, both criteria lead to the conclusion that the segment will operate at LOS E.

Capacity is determined by either Equation 15-12 or Equation 15-13, whichever produces the lower estimate. Note, however, that all adjustment factors for use in these equations are based on a directional flow rate greater than 900 pc/h. Thus, the grade factor will be 1.00 for both ATS and PTSF. The passenger car equivalent for trucks is 1.3 for ATS and 1.00 for PTSF; the passenger car equivalent for RVs is 1.1 for ATS and 1.00 for PTSF.

The adjustment factors for heavy vehicles are as follows:

$$f_{HV,ATS} = \frac{1}{1 + 0.14\,(1.3 - 1) + 0.04\,(1.1 - 1)} = 0.96$$

$$f_{HV,PTSF} = \frac{1}{1 + 0.14\,(1.0 - 1) + 0.04\,(1.0 - 1)} = 1.00$$

and

$$c_{dATS} = 1700 \times f_{g,ATS} \times f_{HV,ATS} = 1{,}700 \times 1.00 \times 0.960 = 1{,}632 \text{ veh/h}$$

$$c_{dPTSF} = 1700 \times f_{g,PTSF} \times f_{HV,PTSF} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

Obviously, the first value holds, and the directional capacity of this facility is 1,632 veh/h. Given the 50/50 directional distribution, the two-way capacity of the segment is 1,632 + 1,632 = 3,264 veh/h. Because this exceeds the limiting capacity of 3,200 pc/h, the directional capacity cannot be achieved with a 50/50 directional distribution. A total two-way capacity of 3,200 pc/h would prevail. In terms of prevailing conditions, the capacity would be 3,200 × 1.00 × 0.960 = 3,072 veh/h. With a 50/50 directional split, this implies a directional capacity of 3,072/2 = 1,536 veh/h.

Discussion

The two-lane highway segment as described is expected to operate poorly, within LOS E. The operation is poor despite the fact that demand is only 842/1,536 = 0.55 of capacity. Both ATS and PTSF are at unacceptable levels (38.6 mi/h and 81.8%, respectively). This solution again highlights the characteristic of two-lane highways of having poor operations at relatively low *v/c* ratios. This segment should clearly be examined for potential improvements.

Given the 50/50 directional split of traffic, results for the second direction would be identical.

EXAMPLE PROBLEM 2: CLASS II HIGHWAY LOS

The Facts

A segment of Class II highway is part of a scenic and recreational route and has the following known characteristics:

- Class II highway
- 1,050 veh/h (both directions)
- 70/30 directional split
- 5% trucks; 7% RVs
- PHF = 0.85
- 10-ft lanes; 2-ft shoulders
- BFFS = 55.0 mi/h
- Rolling terrain
- 10 access points/mi
- 60% no-passing zones

Comments

Computational Steps 3 and 4, which relate to the estimation of average highway speed, will not be included. LOS for Class II highways depends solely on PTSF. The analysis will be conducted for both the 70% direction of flow and the 30% direction of flow. This is accomplished by merely reversing the analysis direction and opposing flows.

Step 1: Input Data

All input data have been summarized above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2. Adjustment factors for lane and shoulder width (Exhibit 15-7) and access points per mile (Exhibit 15-8) are used.

Exhibit 15-7 is entered with 10-ft lanes and 2-ft shoulders. The resulting adjustment is 3.7 mi/h. Exhibit 15-8 is entered with 10 access points/mi. The resulting adjustment is 2.5 mi/h. The FFS is then estimated as follows:

$$FFS = 55.0 - 3.7 - 2.5 = 48.8 \text{ mi/h}$$

Steps 3 and 4

Steps 3 and 4 are not required for Class II highways.

Step 5: Demand Adjustment for PTSF

Equation 15-7 and Equation 15-8 are used to adjust analysis direction and opposing demands to flow rates under equivalent base conditions. With a 70/30 split of traffic, the two demands are as follows:

$$V_{70\%} = V_1 = 1{,}050 \times 0.70 = 735 \text{ veh/h}$$

$$V_{30\%} = V_2 = 1{,}050 \times 0.30 = 315 \text{ veh/h}$$

In this solution, directions will be referred to as 1 and 2. Since both directions are to be analyzed, their position as "analysis direction" and "opposing" will depend on which direction is under study.

Adjustment factors both for grades (Exhibit 15-16) and for heavy vehicles (Exhibit 15-18) are needed. Exhibit 15-16 and Exhibit 15-18 are entered with a directional flow rate of 735/0.85 = 865 veh/h (Direction 1) and 315/0.85 = 371 veh/h (Direction 2). Interpolation is required in both. The following values are obtained:

$f_{g,PTSF}$ = 1.00 (Direction 1); 0.89 (Direction 2)

E_T = 1.0 (Direction 1); 1.6 (Direction 2)

E_R = 1.0 (Direction 1); 1.0 (Direction 2)

The heavy vehicle adjustment factor for both directions is computed with Equation 15-8:

$$f_{HV,PTSF1} = \frac{1}{1 + 0.05(1.00 - 1) + 0.07(1.00 - 1)} = 1.00$$

$$f_{HV,PTSF2} = \frac{1}{1+0.05(1.6-1)+0.07(1.00-1)} = 0.97$$

The adjusted demand flow rates are computed with Equation 15-7:

$$v_{1,PTSF} = \frac{735}{0.85 \times 1.00 \times 1.00} = 865 \text{ pc/h}$$

$$v_{2,PTSF} = \frac{315}{0.85 \times 0.89 \times 0.97} = 429 \text{ pc/h}$$

Step 6: Estimate PTSF

PTSF is estimated with Equation 15-9 and Equation 15-10 with values *a* and *b* taken from Exhibit 15-20 and $f_{np,PTSF}$ taken from Exhibit 15-21.

Exhibit 15-20 is entered with opposing flow rates of 429 pc/h (for Direction 1) and 865 pc/h (for Direction 2). Both values must be interpolated. The resulting values are as follows:

Direction 1: a = –0.0024; b = 0.915

Direction 2: a = –0.0046; b = 0.832

Exhibit 15-21 is entered with the total demand flow rate of 865 + 429 = 1,294 pc/h, a directional split of 70/30, and 60% no-passing zones. Interpolation is required. The factor is the same for both Directions 1 and 2:

$$f_{np,PTSF} = 23.0\%$$

BPTSF is computed with Equation 15-10:

$$BPTSF_1 = 100\left[1 - \exp(-0.0024 \times 865^{0.915})\right] = 68.9\%$$

$$BPTSF_2 = 100\left[1 - \exp(-0.0046 \times 429^{0.832})\right] = 51.0\%$$

The PTSF for each direction is computed with Equation 15-9:

$$PTSF_1 = 68.9 + 23.0\left(\frac{865}{865+429}\right) = 84.3\%$$

$$PTSF_2 = 51.0 + 23.0\left(\frac{429}{429+865}\right) = 58.6\%$$

Step 7

Step 7 is only used for Class III highways.

Step 8: Determine LOS and Capacity

The LOS is determined by comparing the PTSF values obtained with the criteria of Exhibit 15-3. Applying these criteria reveals that Direction 1 operates at LOS D, while Direction 2 operates at LOS C.

By using the adjustment selected for ≥900 veh/h, capacity is computed with Equation 15-13:

$$c_{1,PTSF} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

$$c_{2,PTSF} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

Discussion

The LOS is, at best, somewhat marginal on this two-lane highway segment, based solely on the PTSF.

The value of capacity must be carefully considered. If the directional capacities were expanded to two-way capacities based on the given demand split, the capacity in the 30% direction would imply a two-way capacity well in excess of the 3,200 pc/h limitation for both directions. Therefore, even though a capacity of 1,700 veh/h is possible in the 30% direction, it could not occur with a 70/30 demand split. In this case, the two-way capacity would be limited by the capacity in the 70% direction and would be 1,700/0.70 = 2,429 veh/h. The practical capacity for the 30% direction of flow is actually best estimated as 2,429 – 1,700 or 729 veh/h. Given that the 70/30 directional split holds, when the 30% direction reaches a demand flow rate of 729 veh/h, the opposing direction (the 70% side) would be at its capacity.

EXAMPLE PROBLEM 3: CLASS III HIGHWAY LOS

The Facts

A Class III two-lane highway runs through a rural community in level terrain. It has the following known characteristics:

- Class III highway
- Demand volume = 900 veh/h (both directions)
- 10% trucks; no RVs
- Measured FFS = 40 mi/h
- 12-ft lanes; 6-ft shoulders
- PHF = 0.88
- 80% no-passing zones
- 60/40 directional split
- 40 access points/mi
- Level terrain

Comments

Because this is a Class III highway, LOS will be based on PFFS. Thus, Steps 5 and 6, which relate to the estimation of PTSF, will not be used.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate FFS

A measured FFS is specified: 40 mi/h.

Step 3: Demand Adjustment for ATS

The total demand volume of 900 veh/h must be separated into two directional flows. Since both directions will be evaluated, directions are labeled 1 and 2.

$$V_1 = 900 \times 0.60 = 540 \text{ veh/h}$$

$$V_2 = 900 \times 0.40 = 360 \text{ veh/h}$$

The adjusted demand flow rate in passenger cars per hour under equivalent base conditions is estimated with Equation 15-3. A grade adjustment factor is selected from Exhibit 15-9, and passenger car equivalents for trucks are selected from Exhibit 15-11. Both exhibits are entered with a demand flow rate in vehicles per hour:

$$v_1 = 540 / 0.88 = 614 \text{ veh/h}$$

$$v_2 = 360 / 0.88 = 409 \text{ veh/h}$$

The following values are selected from Exhibit 15-9 and Exhibit 15-11. In all cases, interpolation is required:

Value	Direction 1	Direction 2
$f_{g,ATS}$	1.00	1.00
E_T	1.1	1.3

Then, use of Equation 15-4 gives

$$f_{HV,ATS(1)} = \frac{1}{1 + 0.10(1.1 - 1)} = 0.99$$

$$f_{HV,ATS(2)} = \frac{1}{1 + 0.10(1.3 - 1)} = 0.97$$

Use of Equation 15-3 gives

$$v_{1ATS} = \frac{540}{0.88 \times 1.00 \times 0.99} = 620 \text{ pc/h}$$

$$v_{2ATS} = \frac{360}{0.88 \times 1.00 \times 0.97} = 422 \text{ pc/h}$$

Step 4: Estimate ATS

ATS is estimated with Equation 15-6 with an adjustment factor for no-passing zones taken from Exhibit 15-15. The adjustment factor is based on a 40-mi/h FFS and 80% no-passing zones. Interpolating for an opposing demand flow rate of 422 pc/h (Direction 1) and 620 pc/h (Direction 2) gives the following:

$$f_{np,ATS(1)} = 2.4 \text{ mi/h}$$

$$f_{np,ATS(2)} = 1.6 \text{ mi/h}$$

Then, use of Equation 15-6 gives

$$ATS_1 = 40.0 - 0.00776(620 + 422) - 2.4 = 29.5 \text{ mi/h}$$

$$ATS_2 = 40.0 - 0.00776(422 + 620) - 1.6 = 30.3 \text{ mi/h}$$

Steps 5 and 6

Steps 5 and 6 are not used for Class III highways.

Step 7: Estimate PFFS

The LOS for Class III facilities is based on PFFS achieved, or ATS/FFS. For this segment PFFS is as follows:

$$PFFS_1 = 29.5/40.0 = 73.8\%$$

$$PFFS_2 = 30.3/40.0 = 75.8\%$$

Step 8: Determine LOS and Capacity

From Exhibit 15-3, the LOS for Direction 1 is D, while the LOS for Direction 2 is C. The two values of PFFS are close, but the boundary condition between LOS C and D is 0.75. To be LOS C, PFFS must exceed 0.75, and it is just below the threshold in Direction 1 and just above the threshold in Direction 2.

Capacity is evaluated with adjustment factors for ≥900 pc/h in level terrain. This makes all adjustment factors 1.00 (for ATS). Thus, the capacity in either direction is as follows:

$$c_{1,ATS} = c_{2,ATS} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

The two-way capacity values implied are 1,700/0.60 = 2,833 veh/h (Direction 1) and 1,700/0.40 = 4,250 veh/h (Direction 2). Obviously, the implied two-way capacity is the 2,833 veh/h. Moreover, this suggests that the directional capacity in Direction 2 cannot be achieved with a 60/40 demand split. Rather, the directional capacity in Direction 2 occurs when the capacity in Direction 1 occurs, or 2,833 × 0.40 = 1,133 veh/h.

Discussion

This segment of Class III two-lane highway operates just at the LOS C–D boundary. Depending on the length of the segment and local expectations, this may or may not be acceptable.

EXAMPLE PROBLEM 4: CLASS I HIGHWAY LOS WITH A PASSING LANE

The Facts

The 10-mi segment of the two-lane highway analyzed in Example Problem 1 will be improved with 2-mi passing lanes (one in each direction), both installed at 1.00 mi from the segment's beginning. The segment without a passing lane has already been analyzed, and the results of that analysis are listed below:

- Demand volume = 800 veh/h in each direction
- Demand flow rate (ATS) = 902 pc/h in each direction
- Demand flow rate (PTSF) = 842 pc/h in each directions

- FFS = 53.3 mi/h
- ATS = 38.6 mi/h
- PTSF = 81.8%
- Rolling terrain
- PHF = 0.95

Comments

Both directions will involve the same computations, since the directional distribution is 50/50, and in both cases, the passing lane will start 1.00 mi after the beginning of the segment (of 10 mi) and will end 3.00 mi after the beginning of the segment.

Step 1: Conduct an Analysis Without the Passing Lane

Completed as Example Problem 1.

Step 2: Divide the Segment into Regions

Exhibit 15-35 shows the division of the 6-mi segment into regions. The effective downstream length of the passing lane is selected from Exhibit 15-23 (value is different for ATS and PTSF) for a demand flow rate of 800/0.95 = 842 veh/h.

Exhibit 15-35
Region Lengths for Use in Example Problem 4

To Determine	L_u (mi)	L_{pl} (mi)	L_{de} (mi) Exhibit 15-23	L_d (mi) Equation 15-14
ATS	1.00	2.00	1.7	5.3
PTSF	1.00	2.00	4.7	2.3

Step 3: Determine the PTSF

The PTSF, as affected by the presence of a passing lane, is estimated with Equation 15-15 and an adjustment factor selected from Exhibit 15-26. The adjustment factor $f_{pl,PTSF}$ is 0.62. Then

$$PTSF_{pl} = \frac{PTSF\left[L_u + L_d + f_{pl,PTSF} L_{pl} + \left(\frac{1 + f_{pl,PTSF}}{2}\right) L_{de}\right]}{L_t}$$

$$PTSF_{pl} = \frac{81.8\left[1.0 + 2.3 + (0.62 \times 2.00) + \left(\frac{1 + 0.62}{2}\right) 4.7\right]}{10}$$

$$PTSF_{pl} = \frac{81.8(3.3 + 1.24 + 3.81)}{10} = \frac{81.8 \times 8.35}{10} = 68.3\%$$

Step 4: Determine the ATS

The ATS as affected by the presence of a passing lane is found with Equation 15-17 and an adjustment factor selected from Exhibit 15-28. The adjustment factor selected is 1.11. Then

$$ATS_{pl} = \frac{ATSL_t}{L_u + L_d + \left(\frac{L_{pl}}{f_{pl,ATS}}\right) + \left(\frac{2L_{de}}{1 + f_{pl,ATS}}\right)}$$

$$ATS_{pl} = \frac{38.6 \times 10}{1.00 + 5.3 + \left(\frac{2.00}{1.11}\right) + \left(\frac{2 \times 11.7}{1 + 1.11}\right)}$$

$$ATS_{pl} = \frac{38.6 \times 110}{6.30 + 1.80 + 1.61} = \frac{38.6 \times 110}{9.71} = 39.7 \text{ mi/h}$$

Step 5: Determine the LOS

Exhibit 15-3 shows that the LOS, as determined by PTSF, has improved to D. The LOS determined by ATS remains E. Thus, while PTSF has improved significantly, the ATS has not improved enough to improve the overall LOS, which remains E.

Discussion

Adding a 2-mi passing lane to a 10-mi segment of Class I highway operating at LOS E was insufficient to improve the overall LOS, although the PTSF did improve from 81.8% to 68.3%. It is likely that a longer (or a second) passing lane would be needed to improve the ATS sufficiently to result in LOS C or LOS D.

EXAMPLE PROBLEM 5: TWO-LANE HIGHWAY BICYCLE LOS

A segment of two-lane highway (without passing lanes) is being evaluated for potential widening, realigning, and repaving. Analyze the impacts of the proposed project on the bicycle LOS in the peak direction.

The Facts

The roadway currently has the following characteristics:

- Lane width = 12 ft
- Shoulder width = 2 ft
- Pavement rating = 3 (fair)
- Posted speed limit = 50 mi/h
- Hourly directional volume = 500 veh/h (no growth is expected)
- Percentage of heavy vehicles = 5%
- PHF = 0.90
- No on-highway parking

The proposed roadway design has the following characteristics:

- Lane width = 12 ft
- Shoulder width = 6 ft
- Pavement rating = 5 (very good)
- Posted speed limit = 55 mi/h
- No on-highway parking

Step 1: Gather Input Data

All data needed to perform the analysis are listed above.

Step 2: Calculate the Directional Flow Rate in the Outside Lane

By using the hourly directional volume and the PHF, calculate the directional demand flow rate with Equation 15-24. Because this is a two-lane highway segment without a passing lane, the number of directional lanes N is 1. Because traffic volumes are not expected to grow over the period of the analysis, v_{OL} is the same for both current and future conditions.

$$v_{OL} = \frac{V}{PHF \times N}$$

$$v_{OL} = \frac{500}{0.90 \times 1} = 556 \text{ veh/h}$$

Step 3: Calculate the Effective Width

For current conditions, the hourly directional demand V is greater than 160 veh/h and the paved shoulder width is 2 ft; therefore, Equation 15-27 and Equation 15-28 are used to determine the effective width of the outside lane. Under future conditions, the paved shoulder width will increase to 6 ft; therefore, Equation 15-26 and Equation 15-28 are used.

For current conditions:

$$W_v = W_{OL} + W_s = 12 \text{ ft} + 2 \text{ ft} = 14 \text{ ft}$$

$$W_e = W_v + (\%OHP(2 \text{ ft} + W_s))$$

$$W_e = 14 \text{ ft} + (0 \times (2 \text{ ft} + 2 \text{ ft})) = 14 \text{ ft}$$

Under the proposed design:

$$W_v = W_{OL} + W_s = 12 \text{ ft} + 6 \text{ ft} = 18 \text{ ft}$$

$$W_e = W_v + W_s - 2 \times (\%OHP(2 \text{ ft} + W_s))$$

$$W_e = 18 \text{ ft} + 6 \text{ ft} - 2 \times (0 \times (2 \text{ ft} + 6 \text{ ft})) = 24 \text{ ft}$$

Step 4: Calculate the Effective Speed Factor

Equation 15-30 is used to calculate the effective speed factor. Under current conditions:

$$S_t = 1.1199 \ln(S_p - 20) + 0.8103$$

$$S_t = 1.1199\ln(50-20)+0.8103 = 4.62$$

Under the proposed design:

$$S_t = 1.1199\ln(55-20)+0.8103 = 4.79$$

Step 5: Determine the LOS

Equation 15-31 is used to calculate the bicycle LOS score, which is then used in Exhibit 15-4 to determine the LOS. Under existing conditions:

$$BLOS = 0.507\ln(v_{OL})+0.1999S_t(1+10.38HV)^2$$
$$+7.066(1/P)^2-0.005(W_e)^2+0.057$$

$$BLOS = 0.507\ln(556)+0.1999(4.62)(1+10.38\times 0.05)^2$$
$$+7.066(1/3)^2-0.005(14)^2+0.057$$
$$BLOS = 3.205+2.131+0.785-0.980+0.057 = 5.20$$

Therefore, the bicycle LOS for existing conditions is LOS E. Use of the same process for the proposed design results in the following:

$$BLOS = 0.507\ln(556)+0.1999(4.79)(1+10.38\times 0.05)^2$$
$$+7.066(1/5)^2-0.005(24)^2+0.057$$
$$BLOS = 3.205+2.209+0.283-2.880+0.057 = 2.87$$

The corresponding LOS for the proposed design is LOS C.

Discussion

Although the posted speed would increase as a result of the proposed design, this negative impact on bicyclists would be more than offset by the proposed shoulder widening, as indicated by the improvement from LOS E to LOS C.

5. REFERENCES

Many of these references can be found in the Technical Reference Library in Volume 4.

1. Harwood, D. W., A. D. May, Jr., I. B. Anderson, and A. R. Archilla. *Capacity and Quality of Service of Two-Lane Highways*. Final report, NCHRP Project 3-55(3). Midwest Research Institute, Kansas City, Mo., 1999.
2. Harwood, D. W., I. B. Potts, K. M. Bauer, J. A. Bonneson, and L. Elefteriadou. *Two-Lane Road Analysis Methodology in the Highway Capacity Manual*. Final report, NCHRP Project 20-7(160). Midwest Research Institute, Kansas City, Mo., Sept. 2003.
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APPENDIX A: DESIGN AND OPERATIONAL TREATMENTS

Two-lane highways make up approximately 80% of all paved rural highways in the United States but carry only about 30% of all traffic. For the most part, two-lane highways carry light volumes and experience few operational problems. Some two-lane highways, however, periodically experience significant operational and safety problems brought about by a variety of traffic, geometric, and environmental causes. Such highways may require design or operational improvements to alleviate congestion.

When traffic operational problems occur on two-lane highways, many agencies consider widening to four lanes. Another effective method for alleviating operational problems is to provide passing lanes at intervals in each direction of travel or to provide climbing lanes on steep upgrades. Passing and climbing lanes cannot increase the capacity of a two-lane highway, but they can improve its LOS. Short sections of four-lane highway can function as a pair of passing lanes in opposite directions of travel. Operational analysis procedures for passing and climbing lanes are included in this chapter.

A number of other design and operational treatments are effective in alleviating operational congestion on two-lane highways, including

- Turnouts,
- Shoulder use,
- Wide cross sections,
- Intersection turn lanes, and
- Two-way left-turn lanes.

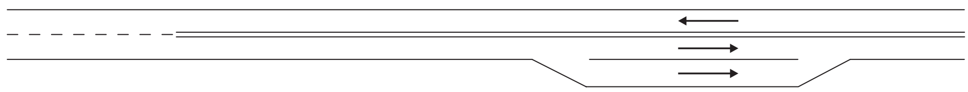
No calculation methodologies are provided in this chapter for these treatments; however, the treatments are discussed below to indicate their potential for improving traffic operations on two-lane highways.

TURNOUTS

A turnout is a widened, unobstructed shoulder area on a two-lane highway that allows slow-moving vehicles to pull out of the through lane so that vehicles following may pass. Turnouts are relatively short, generally less than 625 ft. At a turnout, the driver of a slow-moving vehicle that is delaying one or more following vehicles is expected to pull out of the through lane, allowing the vehicles to pass. The driver of the slow-moving vehicle is expected to remain in the turnout only long enough to allow the following vehicle to pass before returning to the travel lane. When there are only one or two following vehicles, this maneuver can usually be completed smoothly, with no need for the vehicle to stop in the turnout. When there are three or more following vehicles, however, the vehicle in the turnout will generally have to stop to allow all vehicles to pass. In this case, the driver of the slower vehicle is expected to stop before the end of the turnout, so that the vehicle will develop some speed before reentering the lane. Signs inform drivers of the turnout's location and reinforce the legal requirements concerning turnout use.

Turnouts have been used in several countries to provide additional passing opportunities on two-lane highways. In the United States, turnouts have been used extensively in western states. Exhibit 15-A1 illustrates a typical turnout.

Exhibit 15-A1
Typical Turnout Illustrated



Turnouts may be used on nearly any type of two-lane highway that offers limited passing opportunities. To avoid confusing drivers, turnouts and passing lanes should not be intermixed on the same highway.

A single well-designed and well-located turnout can be expected to accommodate 20% to 50% of the number of passes that would occur in a 1.0-mi passing lane in level terrain (*A1, A2*). Turnouts have been found to operate safely, with experts (*A2–A4*) noting that turnout accidents occur at a rate of only 1 per 80,000 to 400,000 users.

SHOULDER USE

The primary purpose of the shoulder on two-lane highways is to provide a stopping and recovery area for disabled or errant vehicles. However, paved shoulders also may be used to increase passing opportunities on two-lane highways.

In some parts of the United States and Canada, if the paved shoulders are adequate, there is a long-standing custom for slower vehicles to move to the shoulder when a vehicle approaches from the rear. The slower vehicle then returns to the travel lane once the passing vehicles has cleared. The custom is regarded as a courtesy and requires little or no sacrifice in speed by either motorist. A few highway agencies encourage drivers of slow-moving vehicles to use the shoulder in this way because it improves the LOS of two-lane highways without the expense of adding passing lanes or widening the highway. On the other hand, there are agencies that discourage this practice because their shoulders are not designed for frequent use by heavy vehicles.

One highway agency in the western United States generally does not permit shoulder use by slow-moving vehicles but designates specific sections on which the shoulder may be used for this purpose. These shoulder segments range in length from 0.2 to 3.0 mi and are identified by traffic signs.

Research (*A1, A2*) has shown that a shoulder-use segment is about 20% as effective in reducing platoons as a passing lane of comparable length.

WIDE CROSS SECTIONS

Two-lane highways with lanes about 50% wider than normal have been used in several European countries as a less expensive alternative to passing lanes. Sweden, for example, built approximately 500 mi of roadways with two 18-ft travel lanes and relatively narrow (3.3-ft) shoulders. The wider lane permits faster vehicles to pass slower vehicles while encroaching only slightly on the opposing lane of traffic. Opposing vehicles must move toward the shoulder to permit such maneuvers. Roadway segments with wider lanes can be provided at

intervals, like passing lanes, to increase passing opportunities on two-lane highways.

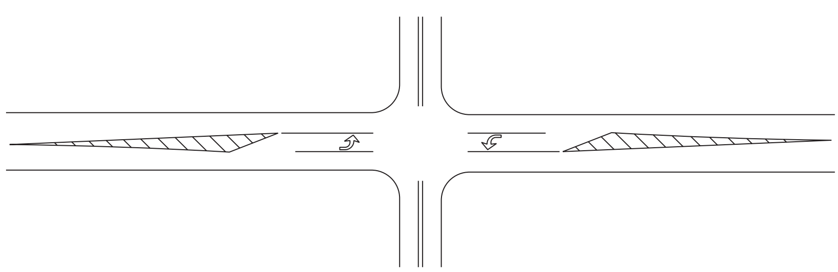
Research has shown that speeds at low traffic volumes tend to increase on wider lanes, but the effect on speeds at higher volumes varies (*A5*). More than 70% of drivers indicated that they appreciate the increased passing opportunities available on the wider lanes. No safety problems have been associated with the wider lanes.

Formal procedures have not yet been developed for evaluating the traffic operational effectiveness of wider lanes in increasing the passing opportunities on a two-lane highway. It is reasonable to estimate the traffic operational performance on a directional two-lane highway segment containing wider lanes as midway between the segment with and without a passing lane of comparable length.

INTERSECTION TURN LANES

Intersection turn lanes are desirable at selected locations on two-lane highways to reduce delays to through vehicles cased by turning vehicles and to reduce turning accidents. Separate right- and left-turn lanes may be provided, as appropriate, to remove turning vehicles from the through travel lanes. Left-turn lanes, in particular, provide a protected location for turning vehicles to wait for an acceptable gap in the opposing traffic stream. This reduces the potential for collisions from the rear and may encourage drivers of left-turning vehicles to wait for an adequate gap in opposing traffic before turning. Exhibit 15-A2 shows a typical two-lane highway with left-turn lanes at an intersection.

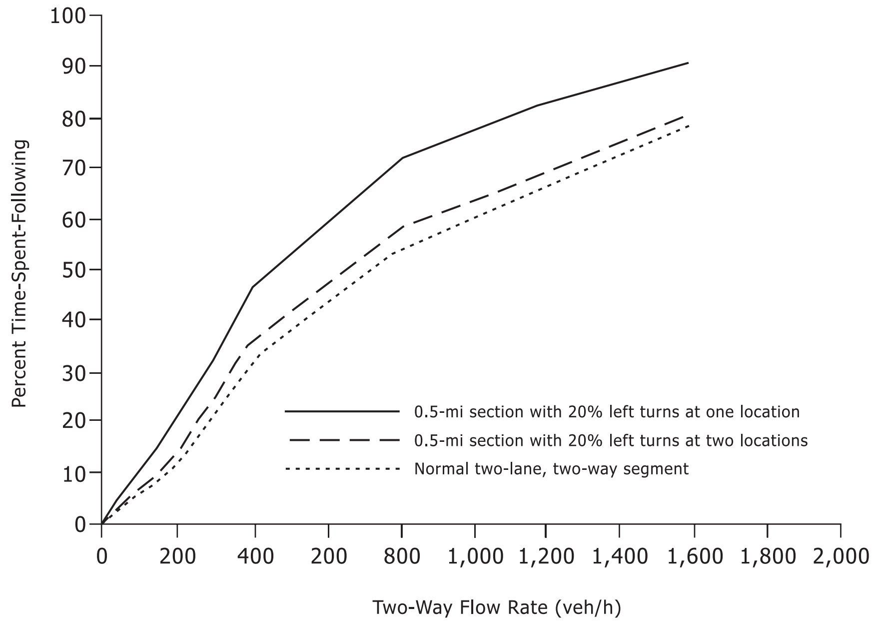
Exhibit 15-A2
Typical Two-Lane Highway Intersection with Left-Turn Lane



Research recommends specific operational warrants for left-turn lanes at intersections on two-lane highways based on the directional volumes and the percentage of left turns (*A6*). The HCM's intersection analysis methodologies can be used to quantify the effects of intersection turn lanes on signalized and unsignalized intersections. There is no methodology, however, for estimating the effect of turn lanes on average highway speed. Modeling of intersection delays shows the relative magnitude of likely effects of turning delays on PTSF (*A7*); the results are shown in Exhibit 15-A3. The top line in the exhibit shows that turning vehicles can increase PTSF substantially over a short road segment. However, when these effects are averaged over a longer road segment, the increase in PTSF is greatly reduced, as indicated by the dashed line in the exhibit. The provision of intersection turn lanes has the potential to minimize these effects.

Exhibit 15-A3
Effect of Turning Delays at Intersections on PTSF

LIVE GRAPH
Click here to view

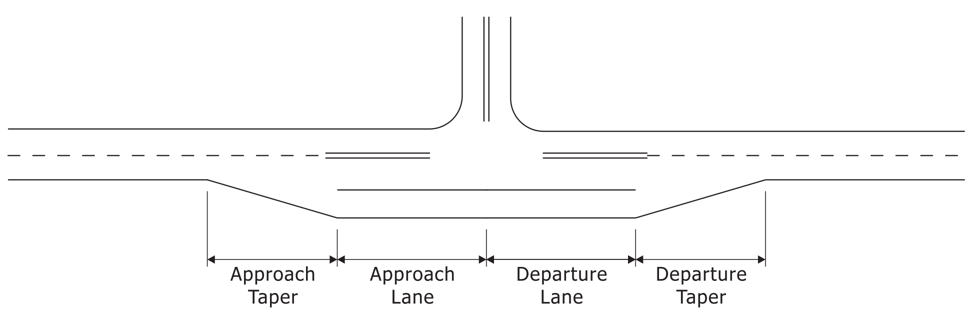


Source: Hoban (*A7*).

Several agencies in the United States provide shoulder bypass lanes at three-leg intersections as a low-cost alternative to a left-turn lane. As shown in Exhibit 15-A4, a portion of the paved shoulder may be marked as a lane for through traffic to bypass vehicles that are slowing or stopped to make a left turn. Bypass lanes may be appropriate for intersections that do not have volumes high enough to warrant a left-turn lane.

The delay benefits of shoulder bypass lanes have not been quantified, but field studies have indicated that 97% of drivers who need to avoid delay will make use of an available shoulder bypass lane. One state has reported a marked decrease in rear-end collisions at intersections where shoulder bypass lanes were provided (*A8*).

Exhibit 15-A4
Typical Shoulder Bypass Lane at a Three-Leg Intersection on a Two-Lane Highway



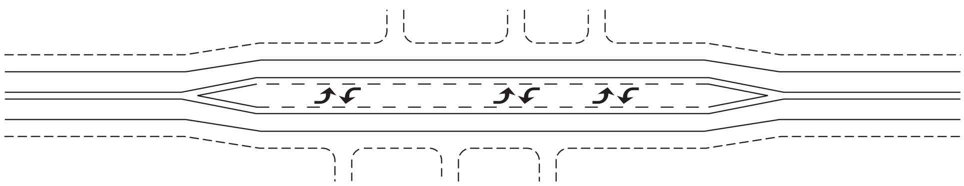
TWO-WAY LEFT-TURN LANES

A two-way left-turn lane (TWLTL) is a paved area in the highway median that extends continuously along a roadway segment and is marked to provide a deceleration and storage area for vehicles traveling in either direction that are making left turns at intersections and driveways.

TWLTLs have been used for many years on urban and suburban streets with high driveway densities and turning demands to improve safety and reduce

delays to through vehicles. TWLTLs can be used on two-lane highways in rural and urban fringe areas to obtain the same types of operational and safety benefits—particularly on Class III two-lane highways. Exhibit 15-A5 illustrates a typical TWLTL.

Exhibit 15-A5
Typical TWLTL on a Two-Lane Highway



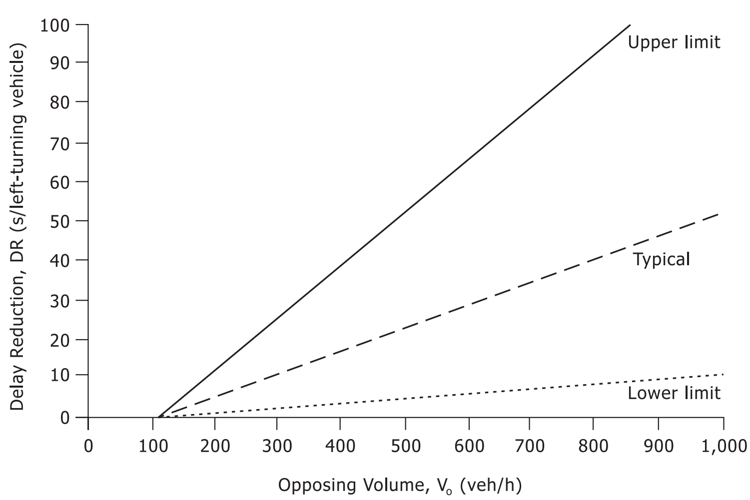
There is no formal methodology for evaluating the traffic operational effectiveness of a TWLTL on a two-lane highway. Research has found that delay reduction provided by a TWLTL depends on both the left-turn demand and the opposing traffic volume (*A2*). Without a TWLTL or other left-turn treatment, vehicles that are slowing or stopped to make a left turn may create delays for following through vehicles. A TWLTL minimizes these delays and makes the roadway segment operate more like two-way and directional segments with 100% no-passing zones. These research results apply to sites that do not have paved shoulders available for following vehicles to bypass turning vehicles. Paved shoulders may alleviate as much of the delay as a TWLTL.

Research has found little delay reduction at rural TWLTL segments with traffic volumes below 300 veh/h in one direction (*A2*). At several low-volume sites, no reduction was observed. The highest delay reduction observed was 3.4 s per left-turning vehicle. Therefore, at low-volume rural sites, TWLTLs should be considered for reducing accidents but should not be expected to improve the operational performance of the highway.

At higher-volume urban fringe sites, greater delay reduction was found with TWLTLs on a two-lane highway. Exhibit 15-A6 shows the expected delay reduction per left-turning vehicle as a function of opposing volume. As the delay reduction increases, a TWLTL can be justified for improving both safety and operations.

Exhibit 15-A6
Estimated Delay Reduction with a TWLTL on a Two-Lane Highway Without Paved Shoulders

LIVE GRAPH
Click here to view



Source: Harwood and St. John (*A2*).

Some of these references can be found in the Technical Reference Library in Volume 4.

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VOLUME 3
INTERRUPTED FLOW

OVERVIEW

Volume 3 of the *Highway Capacity Manual* (HCM) contains eight chapters that present analysis methods for interrupted-flow system elements—that is, roadways and pathways that have fixed causes of periodic delay or interruption to the traffic stream, such as traffic signals and STOP signs. This volume addresses the following types of interrupted-flow system elements:

- *Urban street segments and facilities,* which are portions of roadways that have traffic signals, roundabouts, or STOP-controlled intersections spaced less than 2 mi apart on average;
- *Intersections,* consisting of signalized intersections, two-way STOP-controlled intersections, all-way STOP-controlled intersections, roundabouts, and interchange ramp terminals; and
- *Off-street pedestrian and bicycle facilities* that (*a*) are used only by nonmotorized modes and (*b*) are not considered part of an urban street or transit facility.

VOLUME ORGANIZATION

Urban Street Segments and Facilities

Urban streets typically serve multiple travel modes, in particular the automobile, pedestrian, bicycle, and transit modes. Travelers associated with each of these modes perceive the service provided to them by the urban street in different ways. Design or operational decisions that are intended to improve the service provided to one mode using an urban street can have both adverse and beneficial impacts on the service provided to other modes. The challenge for the analyst is to design and operate an urban street in such a way that all relevant travel modes are reasonably accommodated.

For the purpose of analysis, urban streets are separated into individual elements that are physically adjacent and operate as a single entity in serving travelers. A *point* represents the boundary between links and is represented by an intersection or ramp terminal. A *link* represents a length of roadway between two points. A link and its boundary points are referred to as a *segment.* Multiple contiguous segments can be combined into a single *facility.*

Chapter 17, Urban Street Segments, provides an integrated multimodal methodology for evaluating the quality of service provided to road users traveling along an urban street segment. **Chapter 16, Urban Street Facilities,** provides a similar methodology for evaluating extended lengths of urban streets. In both chapters, level of service (LOS) is reported separately for each mode and is not combined into a single overall LOS. This restriction recognizes that trip purpose, length, and expectation for each mode are different and that their

combination does not produce a meaningful result. This integrated multimodal approach allows analysis of urban streets from a “complete streets” perspective.

Intersections

Five chapters in Volume 3 provide analysis methods for the different kinds of intersections that may be encountered along an urban street.

Chapter 18, Signalized Intersections, describes a methodology for evaluating the capacity and quality of service provided to road users traveling through a signalized intersection. The methodology includes an array of performance measures describing intersection operation for multiple travel modes: automobile, pedestrian, and bicycle. These measures serve as clues for identifying the source of problems and provide insight into the development of effective improvement strategies. The analyst using this methodology is encouraged to consider the full range of measures.

Chapter 19, Two-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing intersections where one street—the *major street*—is uncontrolled, while the other street—the *minor street*—is controlled by STOP signs. These intersections typically occur in one of two configurations: three-leg, where the single minor-street approach is controlled by a STOP sign, and four-leg, where both minor-street approaches are controlled by STOP signs. The methodology is applicable to major streets with up to six lanes, three in each direction. Chapter 19 also provides a methodology for estimating pedestrian delay and LOS in the crossing of major streets at two-way STOP-controlled intersections and at unsignalized midblock crossings.

Chapter 20, All-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing these types of intersections. All-way STOP-controlled intersections require every vehicle to stop at the intersection before proceeding. Because each driver must stop, the decision to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after stopping. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that no vehicles are currently in the intersection and that it is the driver’s turn to proceed.

Chapter 21, Roundabouts, presents concepts and procedures for analyzing modern roundabouts. Roundabouts are intersections with a generally circular shape, characterized by yield on entry and circulation around a central island (counterclockwise in the United States). The methodology can be used to assess the operational performance of existing or planned one-lane or two-lane roundabouts.

Chapter 22, Interchange Ramp Terminals, addresses interchanges with signalized intersections, interchanges with roundabouts, and the impact and operations of adjacent closely spaced intersections. Interchange ramp terminals provide the connection between various highway facilities (e.g., freeway–arterial, arterial–arterial), and thus their efficient operation is essential. In addition, they need to provide adequate capacity to avoid affecting the connecting facilities. The chapter’s methodology can be applied to the operational and planning level

analysis of a broad range of interchange types, including diamond, partial cloverleaf, and single-point urban interchanges.

Off-Street Pedestrian and Bicycle Facilities

Chapter 23, Off-Street Pedestrian and Bicycle Facilities, provides capacity and LOS estimation procedures for the following types of facilities:

- *Walkways*: paved paths, ramps, and plazas that are generally located more than 35 ft from an urban street as well as streets reserved for pedestrian traffic on a full- or part-time basis;
- *Stairways*: staircases that are part of a longer pedestrian facility;
- *Shared-use paths*: paths physically separated from highway traffic for the use of pedestrians, bicyclists, runners, inline skaters, and other users of nonmotorized modes; and
- *Exclusive off-street bicycle paths*: paths physically separated from highway traffic for the exclusive use of bicycles.

On-street pedestrian and bicycle facilities are addressed in the other Volume 3 chapters, particularly Chapters 16–19.

RELATED CHAPTERS

Volume 1

The chapters in Volume 3 assume that the reader is already familiar with the concepts presented in the Volume 1 chapters, in particular the following:

- *Chapter 2, Applications*—types of HCM analysis, types of roadway system elements, and traffic flow characteristics;
- *Chapter 3, Modal Characteristics*—variations in demand, peak and analysis hours, *K*- and *D*-factors, facility types by mode, and interactions between modes;
- *Chapter 4, Traffic Flow and Capacity Concepts*—traffic flow parameters and factors that influence capacity; and
- *Chapter 5, Quality and Level-of-Service Concepts*—performance measures, service measures, and LOS.

Volume 2

Urban streets with two or more lanes in each direction that have traffic signals spaced 2 mi or more apart on average are treated as multilane highways. Chapter 14 provides analysis methods for two-lane highways. Two-lane roadways passing through moderately developed areas, such as small towns or developed recreational areas, are treated as Class III two-lane highways and can be analyzed with the methods given in Chapter 15.

VOLUME 4: APPLICATIONS GUIDE
Methodological Details
29. Urban Street Facilities: Supplemental
30. Urban Street Segments: Supplemental
31. Signalized Intersections: Supplemental
32. STOP-Controlled Intersections: Supplemental
33. Roundabouts: Supplemental
34. Interchange Ramp Terminals: Supplemental
35. Active Traffic Management
Case Studies
Technical Reference Library

Access Volume 4 at www.HCM2010.org

Volume 4

Seven chapters in Volume 4 (accessible at www.HCM2010.org) provide additional information that supplements the material presented in Volume 3. These chapters are as follows:

- *Chapter 29, Urban Street Facilities: Supplemental*—examples of applying alternative tools to situations not addressed by the Chapter 16 method for urban street facilities;
- *Chapter 30, Urban Street Segments: Supplemental*—methods for adjusting traffic demand to account for capacity constraints and midsegment turning movements, analyzing vehicular traffic flow on a segment bounded by signalized intersections, and estimating major-street delay due to midblock turns; a quick-estimation method for evaluating the operation of a coordinated street segment; a description of field measurement techniques; and documentation of the computational engine;
- *Chapter 31, Signalized Intersections: Supplemental*—descriptions of traffic signal concepts and field measurement techniques; details of procedures for calculating capacity, phase duration, delay, and back-of-queue; a quick-estimation method for determining an intersection's critical volume-to-capacity ratio, signal timing, and delay; documentation of the computational engine; and examples of applying alternative tools to situations not addressed by the Chapter 18 method for signalized intersections;
- *Chapter 32, STOP-Controlled Intersections: Supplemental*—methods for determining the potential capacity of two-way STOP-controlled intersections, the operation of two-way STOP-controlled intersections with pedestrian effects, and the operation of all-way STOP-controlled intersections with three-lane approaches; and additional example problems;
- *Chapter 33, Roundabouts: Supplemental*—guidance on lane-use assignment and calibrating the Chapter 21 capacity model;
- *Chapter 34, Interchange Ramp Terminals: Supplemental*—complete solutions to the Chapter 22 example problems, along with additional example problems; and
- *Chapter 35, Active Traffic Management*—descriptions of active traffic management strategies; a discussion of the mechanisms by which they affect demand, capacity, and performance; and general guidance on possible evaluation methods for active traffic management techniques.

The *HCM Applications Guide* in Volume 4 provides three case studies on the analysis of interrupted-flow facilities:

- *Case Study No. 1* illustrates the process of applying HCM techniques to questions relating to the operational and control needs of an intersection located along a roadway in a university town;

- *Case Study No. 2* illustrates the process of applying HCM techniques to the analysis of unsignalized intersections, signalized intersections, and urban streets in the context of a traffic impact analysis; and
- *Case Study No. 5* illustrates the consideration of nonautomobile modes as part of the evaluation of signalized and unsignalized intersections.

These case studies focus on the process of applying the HCM rather than on the details of performing calculations (which are addressed by the example problems in the Volume 3 and supplemental Volume 4 chapters). The case studies' computational results were developed by using HCM2000 methodologies and therefore may not match the results obtained from applying the HCM 2010. However, the process of application is the focus, not the specific computational results.

The Technical Reference Library in Volume 4 contains copies of (or links to) many of the documents referenced in Volume 3 and its supplemental chapters. The Technical Reference Library also provides computational engines (spreadsheets) to assist with the application of the urban streets, signalized intersections, all-way STOP-controlled intersections, and roundabouts methodologies.

CHAPTER 16
URBAN STREET FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 16, Urban Street Facilities, describes an integrated multimodal methodology for evaluating the quality of service provided to road users traveling along an urban street. An urban street is unique among road types because it typically serves multiple travel modes. Four of the more common urban street travel modes include automobile, pedestrian, bicycle, and transit. Travelers associated with each of these modes use different criteria to evaluate the service provided to them when they travel along an urban street. This integrated multimodal approach allows analysts to analyze urban streets from a "complete streets" perspective.

Design or operational decisions that are intended to improve the service provided to one mode can sometimes have an adverse impact on the service provided to another mode. The challenge for the analyst is to design and operate the urban street in such a way that all relevant travel modes are reasonably accommodated. The methodology described in this chapter is intended to assist the analyst in this regard by providing a means to assess the performance of each urban street travel mode.

OVERVIEW OF THE METHODOLOGY

This chapter's methodology is applicable to an urban or suburban street. The street is classified as an arterial or collector with one-way or two-way vehicular traffic flow. The intersections along the street can be signalized or unsignalized.

Analysis Level

Analysis level describes the level of detail used in applying the methodology. Three levels are recognized:

- Operational,
- Design, and
- Planning and preliminary engineering.

The operational analysis is the most detailed application and requires the most information about the traffic, geometric, and signalization conditions. The design analysis also requires detailed information about the traffic conditions and the desired level of service (LOS) as well as information about either the geometric or signalization conditions. The design analysis then seeks to determine reasonable values for the conditions not provided. The planning and preliminary engineering analysis requires only the most fundamental types of information from the analyst. Default values are then used as substitutes for other input data. The subject of analysis level is discussed in more detail in the Applications section of this chapter.

Study Period and Analysis Period

The study period is the time interval represented by the performance evaluation. It consists of one or more consecutive analysis periods. An analysis period is the time interval evaluated by a single application of the methodology.

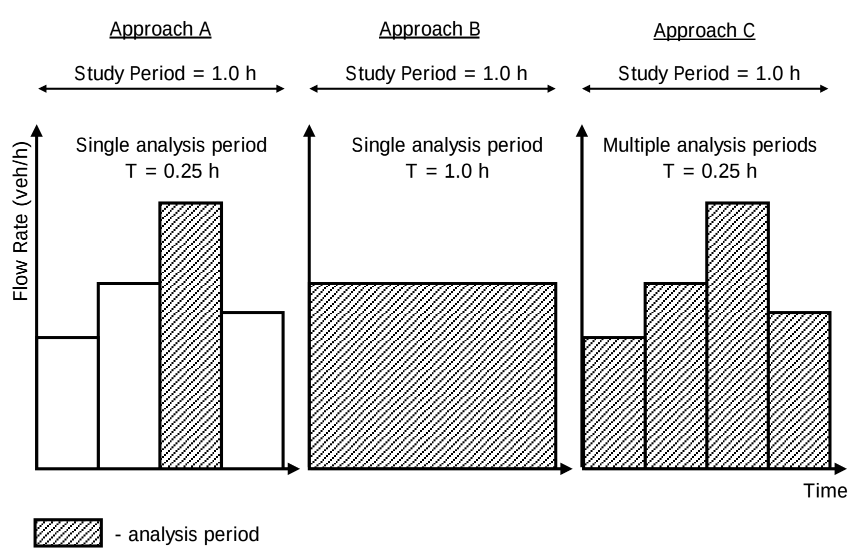
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the duration of the analysis period is in the range of 0.25 to 1 h. The longer durations in this range are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1 h because traffic conditions are not typically steady for long time periods and because the adverse impact of short peaks in traffic demand may not be detected in the evaluation.

If an analysis period of interest has a demand volume that exceeds capacity, then the study period should include an initial analysis period with no initial queue and a final analysis period with no residual queue. This approach provides a more accurate estimate of the delay associated with the congestion.

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be separately reported. In this situation, reporting an average performance for the study period is not encouraged because it may obscure extreme values and suggest acceptable operation when in reality some analysis periods have unacceptable operation.

Exhibit 16-1 demonstrates three alternative approaches an analyst might use for a given evaluation. Note that other alternatives exist and that the study period can exceed 1 h. Approach A is the one that has traditionally been used and, unless otherwise justified, is the one that is recommended for use.

Exhibit 16-1
Three Alternative Study Approaches



Approach A is based on the evaluation of the peak 15-min period during the study period. The analysis period, T, is 0.25 h. The equivalent hourly flow rate in vehicles per hour (veh/h) used for the analysis is based on either a peak 15-min traffic count multiplied by four or a 1-h demand volume divided by the peak hour factor. The former option is preferred whenever traffic counts are available. The peak hour factor equals the hourly count of vehicles divided by four times the peak 15-min count for a common hour interval. It is provided by the analyst or operating agency.

Approach B is based on the evaluation of one 1-h analysis period that is coincident with the study period. The analysis period, *T*, is 1.0 h. The flow rate used is equivalent to the 1-h demand volume (i.e., the peak hour factor is not used). This approach implicitly assumes that the arrival rate of vehicles is constant throughout the period of study. Therefore, the effects of peaking within the hour may not be identified, and the analyst risks underestimating the delay actually incurred.

Approach C uses a 1-h study period and divides it into four 0.25-h analysis periods. This approach accounts for systematic flow rate variation among analysis periods. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay.

Performance Measures

An urban street's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road user group. Performance measures cited in this chapter include automobile travel speed, automobile stop rate, pedestrian space, pedestrian travel speed, pedestrian perception score, bicycle travel speed, bicycle perception score, transit travel speed, and transit passenger perception score.

LOS is also considered a performance measure. It is computed for the automobile, pedestrian, bicycle, and transit travel modes. It is useful for describing street performance to elected officials, policy makers, administrators, or the public. LOS is based on one or more of the performance measures listed in the previous paragraph.

Travel Modes

This chapter describes a separate methodology for evaluating urban street performance from the perspective of motorists, pedestrians, bicyclists, or transit passengers. These methodologies are referred to hereafter as the automobile methodology, pedestrian methodology, bicycle methodology, and transit methodology.

Each methodology consists of a set of procedures for computing the quality of service provided to one mode. Collectively, they can be used to evaluate urban street operation from a multimodal perspective.

Each methodology is focused on the evaluation of the urban street. A methodology for evaluating the segments that make up the street is described in Chapter 17, Urban Street Segments.

The four methodologies described in this chapter are based largely on the products of two National Cooperative Highway Research Program projects (*1*, *2*).

The transit methodology described in this chapter is applicable to the evaluation of passenger service provided by local public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. Nonlocal transit vehicle speed and delay are evaluated by using the automobile methodology.

The phrase *automobile mode,* as used in this chapter, refers to travel by all motorized vehicles that can legally operate on the street, with the exception of local transit vehicles that stop to pick up passengers along the street. Unless explicitly stated otherwise, the word *vehicles* refers to motorized vehicles and includes a mixed stream of automobiles, motorcycles, trucks, and buses.

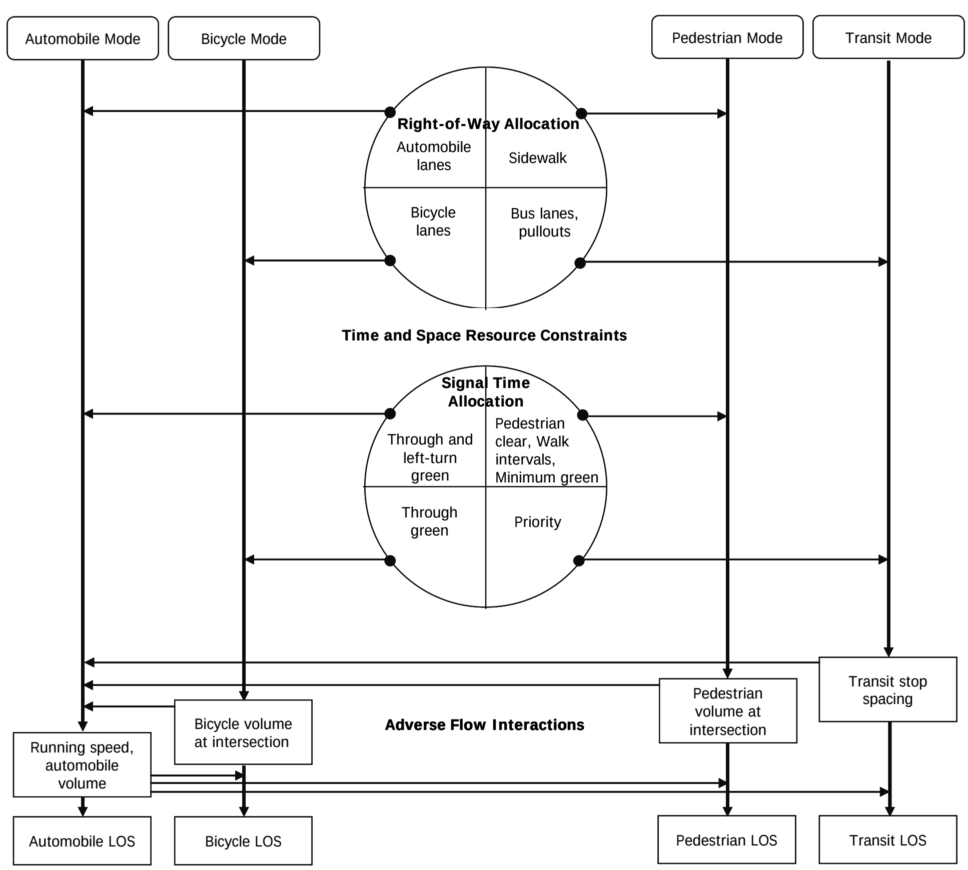
Multimodal Evaluation Framework

The urban street right-of-way is typically shared by multiple travel modes. Travelers associated with the more common modes include motorists, pedestrians, bicyclists, and transit passengers. The factors that influence the quality of service provided to these travelers vary by mode because each mode has a different trip purpose, length, and expectation.

The shared street right-of-way typically requires that the modes operate in close proximity to each other, sometimes even sharing the same portion of the cross section (e.g., a vehicular traffic lane). This arrangement may be workable when the modes are characterized by low demand volumes; however, acceptable operation for moderate to high volumes typically requires the spatial separation of the modes along the street and temporal (i.e., signal) separation at the intersections.

The integrated methodology described in Section 2 can be used to evaluate simultaneously the LOS provided to each travel mode on an urban street. A framework for this evaluation is shown in Exhibit 16-2.

Exhibit 16-2
Integrated Multimodal Evaluation Framework



The framework shown in Exhibit 16-2 illustrates the integrated multimodal evaluation approach supported by the methodology in Section 2. It is important to note that the LOS provided to each travel mode is separately evaluated. The relative importance given to each mode's LOS should be determined by the analyst (or operating agency) and reflect consideration of the subject street's functional class and purpose. The LOS for each mode should *not* be combined into one overall LOS for the street. This restriction recognizes that trip purpose, length, and expectation for each mode are different and that their combination does not produce a meaningful result.

Exhibit 16-2 illustrates how the travel modes compete for limited right-of-way along the street and at the intersections. They also compete for limited signal time at the intersections. For a given right-of-way, the allocation of space to one mode often requires a reduction (or elimination) of space for other modes and a corresponding reduction in their service quality.

The lower part of Exhibit 16-2 illustrates the potential adverse interactions between the automobile mode and the other modes. As the volume or speed of the automobile traffic stream increases, the LOS for the other modes may decrease. In contrast, if bicycle, pedestrian, or transit flows increase, then the LOS for the automobile traffic stream may decrease. In general, changes that alter resource allocation or flow interaction to improve the LOS for one mode may affect the other modes.

URBAN STREET FACILITY DEFINED

For the purpose of analysis, the urban street is separated into individual elements that are physically adjacent and operate as a single entity for the purpose of serving travelers. Two elements are commonly found on an urban street system: points and links. A *point* represents the boundary between links and is usually represented by an intersection or ramp terminal. A *link* represents a length of roadway between two points. A link and its boundary intersections are referred to as a *segment.* An urban street *facility* is a length of roadway that is composed of contiguous urban street segments and is typically functionally classified as an urban arterial or collector street.

Previous editions of this manual have allowed the evaluation of one direction of travel along a facility (even when it served two-way traffic). This approach is retained in this chapter for the analysis of bicycle and transit performance. For the analysis of pedestrian performance, this approach translates into the evaluation of sidewalk and street conditions on one side of the segment.

For the automobile methodology, a segment evaluation considers both directions of travel (when the street serves two-way traffic).

For the analysis of automobile performance, an analysis of only one travel direction (when the street serves two-way traffic) does not adequately recognize the interactions between vehicles at the boundary intersections and their influence on segment operation. For example, the automobile methodology in this edition of the *Highway Capacity Manual* (HCM) explicitly models the platoon formed by the signal at one end of the segment and its influence on the operation of the signal at the other end of the segment. For these reasons, it is important to evaluate both travel directions on a two-way segment.

Facility Length Considerations

Urban arterial and collector streets are designed to accommodate longer trips than local streets. They also have a significant mobility function and support the hierarchy of movement by connecting to streets of higher and lower functional class. An urban street facility with these attributes typically has a length of 1 mi or more in downtown areas and 2 mi or more in other areas. When an urban street facility meets or exceeds this length, average travel speed is a more meaningful indication of facility performance and LOS.

At least one intersection (or ramp terminal) along the facility must have a type of control that can impose on the through movement a legal requirement to stop or yield. A significant change in one or more facility characteristics may indicate the end of one facility and the start of a second facility. These characteristics include cross section features (e.g., number of through lanes, shoulder width, curb presence), annual average daily traffic volume, roadside development density and type, and vehicle speed. One or more of these characteristics will often change significantly when the street crosses an urban-to-suburban area boundary or intersects a freeway interchange.

If a facility assessment is desired for a given travel mode, the analyst will need to evaluate all of the segments that make up the facility for a common travel direction and aggregate the performance measures for each segment to obtain a facility performance estimate.

Facility Versus Segment Analysis Scope

The methodology described in Section 2 is used to evaluate an entire facility; however, for some specific conditions it may not be necessary to evaluate the entire facility. For these conditions, the appropriate segment or intersection chapter methodology may be used alone to evaluate selected segments or intersections. In general, it is up to the analyst to determine the scope of each analysis (i.e., one intersection, one segment, two segments, or all segments on the facility) on the basis of analysis objectives and agency directives.

One condition for which it may be acceptable to evaluate an individual segment or intersection occurs when the segment or intersection is considered to operate in *isolation* from upstream signalized intersections. A segment or intersection that is effectively isolated experiences negligible influence from upstream signalized intersections. Flow on an isolated segment or at an isolated intersection is effectively random over the cycle and without a discernible platoon pattern evident in the cyclic profile of arrivals. These characteristics are more likely to be found when (*a*) the nearest upstream signalized intersection is sufficiently distant from the subject segment or intersection and (*b*) the subject segment or intersection, if signalized, is not coordinated with the upstream signal.

A segment or intersection is sufficiently distant from the nearest upstream signal if an intermediate intersection uses stop or yield control to regulate through traffic on the facility. If there is no intermediate STOP- or YIELD-controlled intersection, then Exhibit 16-3 can be used to obtain an indication of whether a segment or intersection is sufficiently distant from an upstream signal. If the

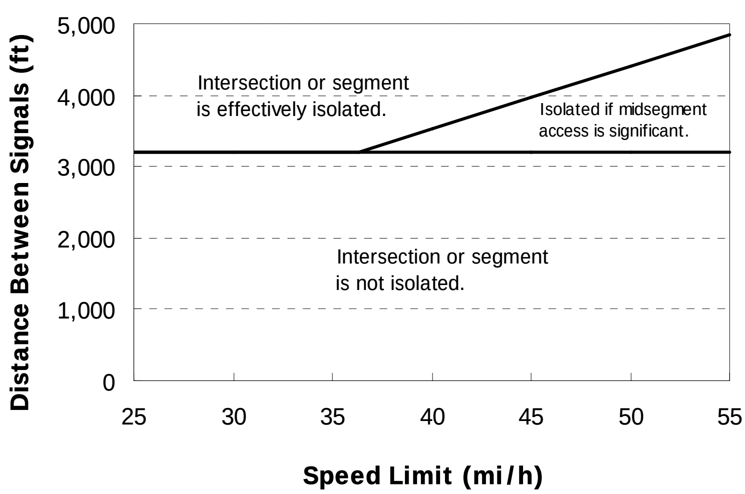
distance between signals is above the trend line, then the subject intersection or segment is likely to operate as effectively isolated (provided that it is not coordinated with the upstream signal).

LOS CRITERIA

This subsection describes the LOS criteria for the automobile, pedestrian, bicycle, and transit modes. The criteria for the automobile mode are different from the criteria used for the nonautomobile modes. Specifically, the automobile mode criteria are based on performance measures that are field-measurable and perceivable by travelers. The criteria for the pedestrian and bike modes are based on scores reported by travelers indicating their perception of service quality. The criteria for the transit mode are based on measured changes in transit patronage due to changes in service quality.

Exhibit 16-3
Signal Spacing Associated with Effectively Isolated Operation

LIVE GRAPH
Click here to view



Automobile Mode

All uses of the word "volume" or the phrase "volume-to-capacity ratio" in this chapter refer to demand volume or demand-volume-to-capacity ratio.

Through-vehicle travel speed is used to characterize vehicular LOS for a given direction of travel along an urban street facility. This speed reflects the factors that influence running time along each link and the delay incurred by through vehicles at each boundary intersection. This performance measure indicates the degree of mobility provided by the facility. The following paragraphs characterize each service level.

LOS A describes primarily free-flow operation. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream. Control delay at the boundary intersections is minimal. The travel speed exceeds 85% of the base free-flow speed.

LOS B describes reasonably unimpeded operation. The ability to maneuver within the traffic stream is only slightly restricted and control delay at the boundary intersections is not significant. The travel speed is between 67% and 85% of the base free-flow speed.

LOS C describes stable operation. The ability to maneuver and change lanes at midsegment locations may be more restricted than at LOS B. Longer queues at

the boundary intersections may contribute to lower travel speeds. The travel speed is between 50% and 67% of the base free-flow speed.

LOS D indicates a less stable condition in which small increases in flow may cause substantial increases in delay and decreases in travel speed. This operation may be due to adverse signal progression, high volume, or inappropriate signal timing at the boundary intersections. The travel speed is between 40% and 50% of the base free-flow speed.

LOS E is characterized by unstable operation and significant delay. Such operations may be due to some combination of adverse progression, high volume, and inappropriate signal timing at the boundary intersections. The travel speed is between 30% and 40% of the base free-flow speed.

LOS F is characterized by flow at extremely low speed. Congestion is likely occurring at the boundary intersections, as indicated by high delay and extensive queuing. The travel speed is 30% or less of the base free-flow speed. Also, LOS F is assigned to the subject direction of travel if the through movement at one or more boundary intersections has a volume-to-capacity ratio greater than 1.0.

Exhibit 16-4 lists the LOS thresholds established for the automobile mode on urban streets.

Exhibit 16-4
LOS Criteria: Automobile Mode

Travel Speed as a Percentage of Base Free-Flow Speed (%)	LOS by Critical Volume-to-Capacity Ratio[a]	
	≤ 1.0	> 1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

Note: [a] The critical volume-to-capacity ratio is based on consideration of the through movement volume-to-capacity ratio at each boundary intersection in the subject direction of travel. The critical volume-to-capacity ratio is the largest ratio of those considered.

Nonautomobile Modes

Historically, this manual has used a single performance measure as the basis for defining LOS. However, research documented in Chapter 5, Quality and Level-of-Service Concepts, indicates that travelers consider a wide variety of factors in assessing the quality of service provided to them. Some of these factors can be described as performance measures (e.g., speed), and others can be described as basic descriptors of the urban street character (e.g., sidewalk width). The methodologies in Chapter 17, Urban Street Segments, and Chapter 18, Signalized Intersections, provide procedures for mathematically combining these factors into a score for the segment or intersection, respectively. This score is then used in this chapter to determine the LOS that is provided for a given direction of travel along a facility.

Exhibit 16-5 lists the range of scores associated with each LOS for the pedestrian travel mode. The LOS for this particular mode is determined by consideration of both the LOS score and the average pedestrian space on the sidewalk. The applicable LOS for an evaluation is determined from the table by

finding the intersection of the row corresponding to the computed score value and the column corresponding to the computed space value.

The association between LOS score and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip along an urban street. The letter "A" was used to represent the "best" quality of service, and the letter "F" was used to represent the "worst" quality of service. "Best" and "worst" were left undefined, allowing respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

Exhibit 16-5
LOS Criteria: Pedestrian Mode

Pedestrian LOS Score	LOS by Average Pedestrian Space (ft^2/p)					
	>60	>40–60	>24–40	>15–24	>8.0–15[a]	≤ 8.0[a]
≤2.00	A	B	C	D	E	F
>2.00–2.75	B	B	C	D	E	F
>2.75–3.50	C	C	C	D	E	F
>3.50–4.25	D	D	D	D	E	F
>4.25–5.00	E	E	E	E	E	F
>5.00	F	F	F	F	F	F

Note: [a] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Exhibit 16-6 lists the range of scores that are associated with each LOS for the bicycle and transit modes. This exhibit is also applicable for determining pedestrian LOS when a sidewalk is not available.

Exhibit 16-6
LOS Criteria: Bicycle and Transit Modes

LOS	LOS Score
A	≤2.00
B	>2.00–2.75
C	>2.75–3.50
D	>3.50–4.25
E	>4.25–5.00
F	>5.00

REQUIRED INPUT DATA

This subsection describes the required input data for the automobile, pedestrian, bicycle, and transit methodologies.

Automobile Mode

This part describes the input data needed for the automobile methodology. The data are listed in Exhibit 16-7 and are identified as "input data elements." For the subject travel direction, these elements must be provided for each segment and for the through-movement group at each boundary intersection.

The last column in Exhibit 16-7 indicates whether the input data are needed for a movement group at a boundary intersection, the overall intersection, or the segment. The input data needed to evaluate the segment are identified in Chapter 17, Urban Street Segments. Similarly, the input data needed to evaluate the boundary intersections are identified in the appropriate chapter (i.e., Chapters 18 to 22).

Exhibit 16-7
Input Data Requirements: Automobile Mode

Data Category	Location	Input Data Element	Basis
Geometric Design	Segment	Segment length	Segment
Other	Segment	Analysis period duration	Facility
Performance Measures	Boundary intersection	Volume-to-capacity ratio	Through-movement group
	Segment	Base free-flow speed	Segment
		Travel speed	Segment

Notes: Through-movement group = one value for the segment through movement at the downstream boundary intersection (inclusive of any turn movements in a shared lane).
Segment = one value or condition for each segment and direction of travel on the facility.
Facility = one value or condition for the facility.

Segment Length

Segment length represents the distance between the boundary intersections that define the segment. The point of measurement at each intersection is the stop line, the yield line, or the functional equivalent in the subject direction of travel. This length is measured along the centerline of the street. If it differs in the two travel directions, then an average length is used. One length is needed for each segment on the facility.

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. Its duration is in the range of 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should use caution in interpreting the results from an analysis period of 1 h or more because the adverse impact of short peaks in traffic demand may not be detected. Also, if the analysis period is other than 15 min, then the peak hour factor should not be used.

The methodology was developed to evaluate conditions in which queue spillback does not affect the performance of a segment or a boundary intersection during the analysis period. If spillback affects performance, the analyst should consider using an alternative analysis tool that is able to model the effect of spillback conditions.

Operational Analysis. A 15-min analysis period should be used for operational analyses. This duration will accurately capture the adverse effects of demand peaks. Any 15-min period of interest can be evaluated with the methodology; however, a complete evaluation should always include an analysis of conditions during the 15-min period that experiences the highest traffic demand during a 24-h period.

If traffic demand exceeds capacity for a given 15-min analysis period, then a multiple-period analysis should be conducted. This type of analysis consists of an evaluation of several consecutive 15-min time periods. The periods analyzed would include an initial analysis period that has no initial queue, one or more periods in which demand exceeds capacity, and a final analysis period that has no residual queue.

When a multiple-period analysis is used, facility performance measures are computed for each analysis period. Averaging performance measures across

multiple analysis periods is not encouraged because it may obscure extreme values.

If a multiple-period analysis is used and the boundary intersections are signalized, then the procedure described in Chapter 18 should be used to guide the evaluation. When a procedure for multiple-period analysis is not provided in the chapter that corresponds to the boundary intersection configuration, the analyst should separately evaluate each period and use the residual queue from one period as the initial queue for the next period.

Planning Analysis. A 15-min analysis period is used for most planning analyses. However, hourly traffic demands are normally produced through the planning process. Thus, when 15-min forecast demands are not available for a 15-min analysis period, a peak hour factor must be used to estimate the 15-min demands for the analysis period. A 1-h analysis period can be used, if appropriate. Regardless of analysis period duration, a single-period analysis is typical for planning applications.

Volume-to-Capacity Ratio

This volume-to-capacity ratio is for the lane group serving the through movement that exits the segment at the downstream boundary intersection. With one exception, a procedure for computing this ratio is described in the appropriate intersection chapter (i.e., Chapters 18 to 22). Chapter 19, Two-Way STOP-Controlled Intersections, does not provide a procedure for estimating the capacity of the uncontrolled through movement, but this capacity can be estimated by using Equation 16-1:

Equation 16-1

$$c_{th} = 1{,}800\ (N_{th} - 1 + p^*_{0,j})$$

where

c_{th} = through-movement capacity (veh/h),

N_{th} = number of through lanes (shared or exclusive) (ln), and

$p^*_{0,j}$ = probability that there will be no queue in the inside through lane.

The probability $p^*_{0,j}$ is computed by using Equation 19-43 in Chapter 19. It is equal to 1.0 if a left-turn bay is provided for left turns from the major street.

One volume-to-capacity ratio is needed for the downstream boundary intersection of each segment on the facility.

Base Free-Flow Speed

The base free-flow speed characterizes the traffic speed on the segment when free-flow conditions exist and speed is uninfluenced by signal spacing. A procedure for determining this speed is described in Chapter 17. One speed is needed for each travel direction on each segment on the facility.

Travel Speed

Travel speed represents the ratio of segment length to through-movement travel time. Travel time is computed as the sum of segment running time and through-movement control delay at the downstream boundary intersection. A

procedure for computing travel speed is described in Chapter 17. One speed is needed for each travel direction of each segment on the facility.

Nonautomobile Modes

This part describes the input data needed for the pedestrian, bicycle, and transit methodologies. The data are listed in Exhibit 16-8 and are identified as "input data elements." They must be separately specified for each direction of travel on the facility. Segment length is defined in the previous part.

Exhibit 16-8 categorizes each input data element by travel mode methodology. An "X" is used to indicate the association between a data element and methodology. A blank cell indicates that the data element is not used as input for the corresponding methodology.

Exhibit 16-8
Input Data Requirements: Nonautomobile Modes

Data Category	Location	Input Data Element	Pedestrian Mode	Bicycle Mode	Transit Mode
Geometric Design	Segment	Segment length	X	X	X
		Presence of a sidewalk	X		
Performance Measures	Segment	Pedestrian space	X		
		Pedestrian travel speed	X		
		Pedestrian LOS score for segment	X		
		Bicycle travel speed		X	
		Bicycle LOS score for segment		X	
		Transit travel speed			X
		Transit LOS score for segment			X

Presence of a Sidewalk

A sidewalk is a paved walkway that is provided at the side of the roadway. It is assumed that pedestrians will walk in the street if a sidewalk is not present. An indication of sidewalk presence is needed for each side of interest for each segment on the facility.

Pedestrian Space

Pedestrian space is a performance measure that describes the average circulation area available to each pedestrian traveling along the sidewalk. A procedure is described in Chapter 17 for estimating this quantity for a given sidewalk. One value is needed for each sidewalk of interest associated with each segment on the facility.

Pedestrian Travel Speed

Pedestrian travel speed represents the ratio of segment length to pedestrian travel time. Travel time is computed as the sum of segment walking time and control delay at the downstream boundary intersection. A procedure for computing this travel speed is described in Chapter 17. One speed is needed for each sidewalk of interest associated with each segment on the facility.

Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is used in the pedestrian methodology to determine facility LOS. It is obtained from the pedestrian

methodology in Chapter 17. One score is needed for each direction of travel of interest for each segment on the facility.

Bicycle Travel Speed

Bicycle travel speed represents the ratio of segment length to bicycle travel time. Travel time is computed as the sum of segment running time and control delay at the downstream boundary intersection. This speed is computed only when a bicycle lane is present on the segment. A procedure for computing this travel speed is described in Chapter 17. One speed is needed for each direction of travel of interest for each segment on the facility.

Bicycle LOS Score for Segment

The bicycle LOS score for the segment is used in the bicycle methodology to estimate facility LOS. It is obtained from the bicycle methodology in Chapter 17. One score is needed for each direction of travel of interest for each segment on the facility.

Transit Travel Speed

Transit travel speed represents the ratio of segment length to transit travel time. Travel time is computed as the sum of segment running time and control delay at the downstream boundary intersection. A procedure for computing this travel speed is described in Chapter 17. One speed is needed for each direction of travel of interest for each segment on the facility.

Transit LOS Score for Segment

The transit LOS score for the segment is used in the transit methodology to estimate facility LOS. It is obtained from the transit methodology in Chapter 17. One score is needed for each direction of travel of interest for each segment on the facility.

SCOPE OF THE METHODOLOGY

Four methodologies are presented in this chapter. One methodology is provided for each of the automobile, pedestrian, bicycle, and transit modes. This section identifies the conditions for which each methodology is applicable.

- **Signalized and two-way STOP-controlled boundary intersections.** All methodologies can be used to evaluate facility performance with signalized or two-way STOP-controlled boundary intersections. In the latter case, the cross street is STOP controlled. The automobile methodology can also be used to evaluate performance with all-way STOP- or YIELD-controlled (e.g., roundabout) boundary intersections.
- **Arterial and collector streets.** The four methodologies were developed with a focus on arterial and collector street conditions. If a methodology is used to evaluate a local street, then the performance estimates should be carefully reviewed for accuracy.
- **Steady flow conditions.** The four methodologies are based on the analysis of steady traffic conditions and, as such, are not well suited to the

evaluation of unsteady conditions (e.g., congestion, queue spillback, signal preemption).

- **Target road users.** Collectively, the four methodologies were developed to estimate the LOS perceived by automobile drivers, pedestrians, bicyclists, and transit passengers. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers, automobile passengers, delivery truck drivers, or recreational vehicle drivers). However, it is likely that the perceptions of these other road users are reasonably well represented by the road users for whom the methodologies were developed.
- **Target travel modes.** The automobile methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The pedestrian, bicycle, and transit methodologies address travel by walking, bicycle, and transit vehicle, respectively. The transit methodology is limited to the evaluation of public transit vehicles operating in mixed or exclusive traffic lanes and stopping along the street. The methodologies are not designed to evaluate the performance of other travel means (e.g., grade-separated rail transit, golf carts, or motorized bicycles).
- **Influences in the right-of-way.** A road user's perception of quality of service is influenced by many factors inside and outside of the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside of the right-of-way (e.g., buildings, parking lots, scenery, or landscaped yards) that might influence a traveler's perspective. This approach was followed because factors outside of the right-of-way are not under the direct control of the agency operating the street.
- **Mobility focus for automobile methodology.** The automobile methodology is intended to facilitate the evaluation of mobility. Accessibility to adjacent properties by way of automobile is not directly evaluated with this methodology. Regardless, a segment's accessibility should also be considered when its performance is evaluated, especially if the street is intended to provide such access. Oftentimes, factors that favor mobility reflect minimal levels of access and vice versa.
- **"Typical pedestrian" focus for pedestrian methodology.** The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. As such, the performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to the Americans with Disabilities Act requirements. For this reason, they should not be considered as a substitute for a formal compliance assessment of a pedestrian facility.

LIMITATIONS OF THE METHODOLOGY

The urban street facility methodology uses the performance measures estimated by the segment and intersection methodologies in Chapters 17 to 22. As such, it incorporates the limitations of these methodologies (which are identified in the respective segment or intersection chapter).

2. METHODOLOGY

OVERVIEW

This section describes four methodologies for evaluating the performance of an urban street facility. Each methodology addresses one possible travel mode within the street right-of-way. Analysts should choose the combination of methodologies that are appropriate for their analysis needs.

A complete evaluation of facility operation includes the separate examination of performance for all relevant travel modes for each travel direction. The performance measures associated with each mode and travel direction are assessed independently of one another. They are not mathematically combined into a single indicator of facility performance. This approach ensures that all performance impacts are considered on a mode-by-mode and direction-by-direction basis.

The focus of each methodology in this chapter is the facility. Methodologies for quantifying the performance of a segment or boundary intersection are described in other chapters (i.e., Chapters 17 to 22).

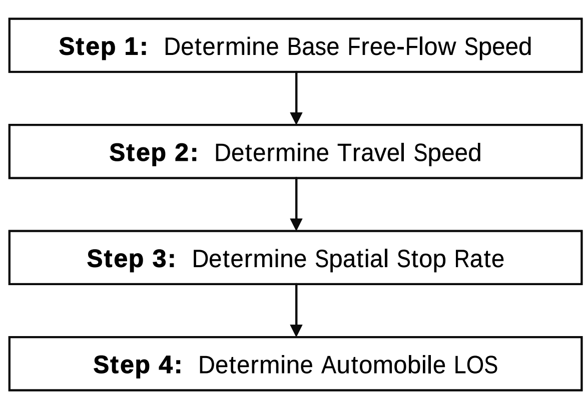
AUTOMOBILE MODE

This subsection provides an overview of the methodology for evaluating urban street facility performance from the motorist perspective. Each travel direction along the facility is separately evaluated. *Unless otherwise stated, all variables are specific to the subject direction of travel.*

The methodology is focused on the analysis of facilities with signalized, two-way STOP, all-way STOP, or roundabout boundary intersections. The signalized intersection can be an interchange ramp terminal.

Exhibit 16-9 illustrates the calculation framework of the automobile methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. The calculations are described more fully in the remainder of this subsection.

Exhibit 16-9
Automobile Methodology for Urban Street Facilities



Step 1: Determine Base Free-Flow Speed

The base free-flow speed for the facility is the basis for LOS determination. It is determined for each segment by using the procedures described in Chapter 17, Urban Street Segments. The base free-flow speed for the facility is calculated by using Equation 16-2:

$$S_{fo,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{fo,i}}}$$

Equation 16-2

where

$S_{fo,F}$ = base free-flow speed for the facility (mi/h),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$S_{fo,i}$ = base free-flow speed for segment i (mi/h).

Step 2: Determine Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed for the through-vehicle traffic stream that reflects the running speed along the street for through vehicles and any delay they may incur at the boundary intersections. The travel speed for through vehicles is determined for each segment by using the procedures described in Chapter 17. The travel speed for the facility is calculated by using Equation 16-3:

$$S_{T,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{T,seg,i}}}$$

Equation 16-3

where $S_{T,F}$ is the travel speed for the facility (mi/h), $S_{T,seg,i}$ is the travel speed of through vehicles for segment i (mi/h), and other variables are as previously defined.

Step 3: Determine Spatial Stop Rate

The spatial stop rate for the facility is the ratio of stop count to facility length. It relates the number of full stops incurred by the average through vehicle to the distance traveled. The spatial stop rate for through vehicles is determined for each segment by using the procedures described in Chapter 17. The spatial stop rate for the facility is calculated by using Equation 16-4:

Equation 16-4

$$H_F = \frac{\sum_{i=1}^{m} H_{seg,i}\ L_i}{\sum_{i=1}^{m} L_i}$$

where H_F is the spatial stop rate for the facility (stops/mi), $H_{seg,i}$ is the spatial stop rate for segment i (stops/mi), and other variables are as previously defined.

The spatial stop rate from Equation 16-4 can be used to estimate an automobile traveler perception score for the facility if desired. The equations in Step 10 of Chapter 17 are used for this purpose. The value of H_F would be substituted for H_{seg} in each equation. Similarly, the proportion of intersections with a left-turn lane P_{LTL} would be calculated for the entire facility and this one value used in each equation.

Step 4: Determine Automobile LOS

LOS is determined for both directions of travel along the facility. Exhibit 16-4 lists the LOS thresholds established for this purpose. As indicated in this exhibit, LOS is defined by travel speed, expressed as a percentage of the base free-flow speed. The base free-flow speed is computed in Step 1 and the travel speed is computed in Step 2.

The footnote to Exhibit 16-4 indicates that volume-to-capacity ratio for the through movement at the downstream boundary intersections is also relevant to the determination of facility LOS. This footnote indicates that LOS F is assigned to the subject direction of travel if a volume-to-capacity ratio greater than 1.0 exists for the through movement at one or more boundary intersections.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

PEDESTRIAN MODE

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to pedestrians.

Urban street facility performance from a pedestrian perspective is separately evaluated for each side of the street. *Unless otherwise stated, all variables identified in this section are specific to the subject side of the street.*

The methodology is focused on the analysis of facilities with either signal-controlled or two-way STOP-controlled boundary intersections. This edition of the HCM does not include a procedure for evaluating a facility's performance when a boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

The pedestrian methodology is applied through a series of four steps that culminate in the determination of the facility LOS. These steps are illustrated in Exhibit 16-10.

Exhibit 16-10
Pedestrian Methodology for Urban Street Facilities

Step 1: Determine Pedestrian Space

Step 2: Determine Pedestrian Travel Speed

Step 3: Determine Pedestrian LOS Score

Step 4: Determine Pedestrian LOS

Concepts

The methodology provides a variety of measures for evaluating facility performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip along the facility. One measure is the LOS score. This score is an indication of the typical pedestrian's perception of the overall facility travel experience. A second measure is the average speed of pedestrians traveling along the facility.

A third measure is based on the concept of "circulation area." It represents the average amount of sidewalk area available to each pedestrian walking along the facility. A larger area is more desirable from the pedestrian perspective. Exhibit 16-11 provides a qualitative description of pedestrian space that can be used to evaluate sidewalk performance from a circulation-area perspective.

Exhibit 16-11
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p)		
Random Flow	**Platoon Flow**	**Description**
>60	>530	Ability to move in desired path, no need to alter movements
>40–60	>90–530	Occasional need to adjust path to avoid conflicts
>24–40	>40–90	Frequent need to adjust path to avoid conflicts
>15–24	>23–40	Speed and ability to pass slower pedestrians restricted
>8–15	>11–23	Speed restricted, very limited ability to pass slower pedestrians
≤ 8	≤ 11	Speed severely restricted, frequent contact with other users

The first two columns in Exhibit 16-11 indicate a sensitivity to flow condition. Random pedestrian flow is typical of most facilities. Platoon flow is appropriate for facilities made up of shorter segments (e.g., in downtown areas) with signalized boundary intersections.

Step 1: Determine Pedestrian Space

Pedestrians are sensitive to the amount of space separating them from other pedestrians and obstacles as they walk along a sidewalk. Average pedestrian space is an indicator of facility performance for travel in a sidewalk. This step is applicable only when the sidewalk exists on the subject side of the street.

The pedestrian space is determined for each segment by using the procedures described in Chapter 17, Urban Street Segments. The pedestrian space for the facility is calculated by using Equation 16-5:

Equation 16-5

$$A_{p,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{A_{p,i}}}$$

where

$A_{p,F}$ = pedestrian space for the facility (ft²/p),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$A_{p,i}$ = pedestrian space for segment i (ft²/p).

The pedestrian space for the facility reflects the space provided on the sidewalk along the segment. It does not consider the corner circulation area or the crosswalk circulation area at the intersections. Regardless, the analyst should verify that the intersection corners and crosswalks adequately accommodate pedestrians.

Step 2: Determine Pedestrian Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed of pedestrians that reflects their walking speed along the sidewalk and any delay they may incur at the boundary intersections. The travel speed for pedestrians is determined for each segment by using the procedures described in Chapter 17. The pedestrian travel speed for the facility is calculated by using Equation 16-6:

Equation 16-6

$$S_{Tp,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{Tp,seg,i}}}$$

where $S_{Tp,F}$ is the travel speed of through pedestrians for the facility (ft/s), $S_{Tp,seg,i}$ is the travel speed of through pedestrians for segment i (ft/s), and other variables are as previously defined.

In general, a travel speed of 4.0 ft/s or more is considered desirable, and a speed of 2.0 ft/s or less is considered undesirable.

Step 3: Determine Pedestrian LOS Score

The pedestrian LOS score for the facility is computed in this step. It represents a length-weighted average of the pedestrian LOS scores for the individual segments that make up the facility. The segment scores are determined by using the procedures described in Chapter 17. The score for the facility is calculated by using Equation 16-7:

$$I_{p,F} = \frac{\sum_{i=1}^{m} I_{p,seg,i}\ L_i}{\sum_{i=1}^{m} L_i}$$

Equation 16-7

where

$I_{p,F}$ = pedestrian LOS score for the facility, and

$I_{p,seg,i}$ = pedestrian LOS score for segment i.

Other variables are as previously defined.

Step 4: Determine Pedestrian LOS

The pedestrian LOS for the facility is determined by using the pedestrian LOS score from Step 3 and the average pedestrian space from Step 1. These two performance measures are compared with their respective thresholds in Exhibit 16-5 to determine the LOS for the specified direction of travel along the subject facility. If the sidewalk does not exist and pedestrians are relegated to walking in the street, then LOS is determined by using Exhibit 16-6 because the pedestrian space concept does not apply.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

BICYCLE MODE

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to bicyclists.

Urban street facility performance from a bicyclist perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.*

The methodology is focused on the analysis of a facility with either signal-controlled or two-way STOP-controlled boundary intersections. This edition of the HCM does not include a procedure for evaluating a facility's performance when a boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

The bicycle methodology is applied through a series of three steps that culminate in the determination of the facility LOS. These steps are illustrated in Exhibit 16-12.

Exhibit 16-12
Bicycle Methodology for Urban Street Facilities

Step 1: Determine Bicycle Travel Speed

↓

Step 2: Determine Bicycle LOS Score

↓

Step 3: Determine Bicycle LOS

Step 1: Determine Bicycle Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed of bicycles that reflects their running speed along the street and any delay they may incur at the boundary intersections. The travel speed for bicycles is determined for each segment by using the procedures described in Chapter 17. The bicycle travel speed for the facility is calculated by using Equation 16-8:

Equation 16-8

$$S_{Tb,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{Tb,seg,i}}}$$

where

$S_{Tb,F}$ = travel speed of through bicycles for the facility (mi/h),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$S_{Tb,seg,i}$ = travel speed of through bicycles for segment i (mi/h).

Step 2: Determine Bicycle LOS Score

The bicycle LOS score for the facility is computed in this step. It represents a length-weighted average of the bicycle LOS scores for the individual segments that make up the facility. The segment scores are determined by using the procedures described in Chapter 17. The score for the facility is calculated by using Equation 16-9:

Equation 16-9

$$I_{b,F} = \frac{\sum_{i=1}^{m} I_{b,seg,i}\ L_i}{\sum_{i=1}^{m} L_i}$$

where $I_{b,F}$ is the bicycle LOS score for the facility, $I_{b,seg,i}$ is the bicycle LOS score for segment i, and other variables are as previously defined.

Step 3: Determine Bicycle LOS

The bicycle LOS for the facility is determined by using the bicycle LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 16-6 to determine the LOS for the specified direction of travel along the subject facility.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

TRANSIT MODE

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to transit passengers.

Urban street facility performance from a transit passenger perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.*

The methodology is applicable to public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. Procedures for estimating transit vehicle performance on grade-separated or non-public-street rights-of-way, along with procedures for estimating origin–destination service quality, are provided in the *Transit Capacity and Quality of Service Manual* (*3*).

The transit methodology is applied through a series of three steps that culminate in the determination of facility LOS. These steps are illustrated in Exhibit 16-13. If multiple routes exist on the segment, then each route is evaluated by using a separate application of this methodology.

Exhibit 16-13
Transit Methodology for Urban Street Facilities

Step 1: Determine Transit Travel Speed

↓

Step 2: Determine Transit LOS Score

↓

Step 3: Determine Transit LOS

Step 1: Determine Transit Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed of transit vehicles that reflects their running speed along the street and any delay they may incur at the boundary intersection. The travel speed for a transit vehicle is determined for each segment by using the procedures described in Chapter 17. The transit travel speed for the facility is calculated by using Equation 16-10:

Equation 16-10

$$S_{Tt,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{Tt,seg,i}}}$$

where

$S_{Tt,F}$ = travel speed of transit vehicles for the facility (mi/h),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$S_{Tt,seg,i}$ = travel speed of transit vehicles for segment i (mi/h).

Step 2: Determine Transit LOS Score

The transit LOS score for the facility is computed in this step. It represents a length-weighted average of the transit LOS score for the individual segments that make up the facility. The segment scores are determined by using the procedures described in Chapter 17. The score for the facility is calculated by using Equation 16-11:

Equation 16-11

$$I_{t,F} = \frac{\sum_{i=1}^{m} I_{t,seg,i}\ L_i}{\sum_{i=1}^{m} L_i}$$

where $I_{t,F}$ is the transit LOS score for the facility, $I_{t,seg,i}$ is the transit LOS score for segment i, and other variables are as previously defined.

Step 3: Determine Transit LOS

The transit LOS for the facility is determined by using the transit LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 16-6 to determine the LOS for the specified direction of travel along the subject facility.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

3. APPLICATIONS

TYPES OF ANALYSIS

The automobile, pedestrian, bicycle, and transit methodologies described in this chapter can each be used in three types (or levels) of analysis. These analysis levels are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis level are described in the subsequent parts of this subsection.

Operational Analysis

Each of the methodologies is most easily applied at an operational level of analysis. At this level, all traffic, geometric, and signalization conditions are specified as input variables by the analyst. These input variables are used in the methodology to compute various performance measures.

Design Analysis

The design level of analysis has two variations. Both variations require the specification of traffic conditions and target levels for a specified set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

The second variation of the design level requires the additional specification of the geometric conditions. The methodology is then applied by using an iterative approach in which alternative signalization conditions are evaluated.

The objective of the design analysis is to identify the alternatives that operate at the target level of the specified performance measures (or provide a better level of performance). The analyst may then recommend the "best" design alternative after consideration of the full range of factors.

Planning and Preliminary Engineering Analysis

The planning and preliminary engineering level of analysis is intended to provide an estimate of the LOS for either a proposed facility or an existing facility in a future year. This level of analysis may also be used to size the overall geometrics of a proposed facility.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose are provided in Chapters 17 to 22.

USE OF ALTERNATIVE TOOLS

Chapter 29, Urban Street Facilities: Supplemental, includes a set of examples to illustrate the use of alternative tools to address the stated limitations of this chapter and Chapter 17, Urban Street Segments. Specifically, these examples are used to illustrate (*a*) the application of deterministic tools to optimize the signal

timing, (*b*) the effect of using a roundabout as a segment boundary, (*c*) the effect of midsegment parking maneuvers on facility operation, and (*d*) the use of simulated vehicle trajectories to evaluate the proportion of time that the back of the queue on the minor-street approach to a two-way STOP-controlled intersection exceeds a specified distance from the stop line.

GENERALIZED DAILY SERVICE VOLUMES FOR URBAN STREET FACILITIES

Generalized daily service volume tables provide a means to assess a large number of urban streets in a region or jurisdiction quickly to determine which facilities need to be assessed more carefully (by using operational analysis) to ameliorate existing or pending problems.

To build a generalized daily service volume table for urban street facilities, a number of simplifying assumptions must be made. The assumptions made here include the following:

- All segments of the facility have the same number of through lanes (one, two, or three) in each direction;
- Only traffic signal control is used along the facility (i.e., no roundabouts or all-way STOP-controlled intersections exist);
- The traffic signals are coordinated and semi-actuated, the arrival type is 4, the traffic signal cycle time C is 120 s, and the weighted average green-to-cycle-length (g/C) ratio for through movements (defined below) is 0.45;
- Exclusive left-turn lanes with protected left-turn phasing and adequate queue storage are provided at each signalized intersection, and no exclusive right-turn lanes are provided;
- At each traffic signal, 10% of the traffic on the urban street facility turns left and 10% turns right;
- The peak hour factor is 0.92;
- The facility length is 2 mi, and no restrictive medians exist along the facility; and
- The base saturation flow rate s_o is 1,900 passenger cars per hour per lane (pc/h/ln).

The weighted average g/C ratio of an urban street is the average of the critical intersection through g/C ratio and the average of all the other g/C ratios for the urban street. For example, if there are four signals with a through g/C ratio of 0.50 and one signal with a through g/C ratio of 0.40, the weighted average g/C ratio for the urban street is 0.45. The weighted g/C ratio takes into account the adverse effect of the critical intersection and the overall quality of flow for the urban street.

Generalized daily service volumes are provided in Exhibit 16-14 for urban street facilities with posted speeds of 30 and 45 mi/h; two, four, or six lanes (both directions); and six combinations of the K-factor and D-factor. To use this table, analysts must select a combination of K and D appropriate for their locality.

The 30-mi/h values further assume an average traffic signal spacing of 1,050 ft and 20 access points/mi, while the 45-mi/h values assume an average traffic signal spacing of 1,500 ft and 10 access points/mi.

Exhibit 16-14
Generalized Daily Service Volumes for Urban Street Facilities (1,000 veh/day)

K-Factor	*D*-Factor	Two-Lane Streets LOS B	LOS C	LOS D	LOS E	Four-Lane Streets LOS B	LOS C	LOS D	LOS E	Six-Lane Streets LOS B	LOS C	LOS D	LOS E
Posted Speed = 30 mi/h													
0.09	0.55	NA	5.9	15.4	19.9	NA	11.3	31.4	37.9	NA	16.3	46.4	54.3
	0.60	NA	5.4	14.1	18.3	NA	10.3	28.8	34.8	NA	15.0	42.5	49.8
0.10	0.55	NA	5.3	13.8	17.9	NA	10.1	28.2	34.1	NA	14.7	41.8	48.9
	0.60	NA	4.8	12.7	16.4	NA	9.3	25.9	31.3	NA	13.5	38.3	44.8
0.11	0.55	NA	4.8	12.6	16.3	NA	9.2	25.7	31.0	NA	13.4	38.0	44.5
	0.60	NA	4.4	11.5	14.9	NA	8.4	23.5	28.4	NA	12.2	34.8	40.8
Posted Speed = 45 mi/h													
0.09	0.55	NA	10.3	18.6	19.9	NA	21.4	37.2	37.9	NA	31.9	54.0	54.3
	0.60	NA	9.4	17.1	18.3	NA	19.6	34.1	34.8	NA	29.2	49.5	49.8
0.10	0.55	NA	9.3	16.8	17.9	NA	19.3	33.5	34.1	NA	28.7	48.6	48.9
	0.60	NA	8.5	15.4	16.4	NA	17.7	30.7	31.3	NA	26.3	44.5	44.8
0.11	0.55	NA	8.4	15.3	16.3	NA	17.5	30.5	31.0	NA	26.1	44.2	44.4
	0.60	NA	7.7	14.0	14.9	NA	16.1	27.9	28.4	NA	23.9	40.5	40.7

Notes: NA = not applicable; LOS cannot be achieved with the stated assumptions.
General assumptions include no roundabouts or all-way STOP-controlled intersections along the facility; coordinated, semi-actuated traffic signals; arrival type 4; 120-s cycle time; protected left-turn phases; 0.45 weighted average *g/C* ratio; exclusive left-turn lanes with adequate queue storage provided at traffic signals; no exclusive right-turn lanes provided; no restrictive median; 2-mi facility length; 10% of traffic turns left and 10% turns right at each traffic signal; peak hour factor = 0.92; and base saturation flow rate = 1,900 pc/h/ln.
Additional assumptions for 30-mi/h facilities: signal spacing = 1,050 ft and 20 access points/mi.
Additional assumptions for 45-mi/h facilities: signal spacing = 1,500 ft and 10 access points/mi.

Exhibit 16-14 is provided for general planning use and should *not* be used to analyze any specific urban street facility or to make final decisions on important design features. A full operational analysis using this chapter's methodology is required for such specific applications.

The exhibit is useful, however, in evaluating the overall performance of a large number of urban streets within a jurisdiction, as a first pass to determine where problems might exist or arise, or to determine where improvements might be needed. Any urban street identified as likely to experience problems or need improvement, however, should then be subjected to a full operational analysis before any decisions on implementing specific improvements are made.

Daily service volumes are strongly affected by the *K*- and *D*-factors chosen as typical for the analysis. It is important that the values used for the facilities under study be reasonable. Also, if any characteristic is significantly different from the typical values used to develop Exhibit 16-14, particularly the weighted average *g*/*C* ratio and traffic signal spacing, the values taken from this exhibit will not be representative of the study facilities. In such cases, analysts are advised to develop their own generalized service volume tables by using representative local values or to proceed to a full operational analysis.

ACTIVE TRAFFIC MANAGEMENT STRATEGIES

Active traffic management (ATM) consists of the dynamic and continuous monitoring and control of traffic operations on a facility to improve facility performance. Examples of ATM measures on urban streets include congestion pricing zones, adaptive/responsive signal control, demand metering, changeable

message signs, incident response, and work zone management. ATM measures can influence both the nature of the demand for the facility and the ability of the facility to deliver the capacity tailored to serve the demand. ATM measures can boost facility performance to the same extent as adding a conventional lane of capacity.

Other advanced design and management measures not included in the definition of ATM can also significantly boost facility performance. These non-ATM measures include lane treatments (bus lanes, bus streets, and reversible lanes), advanced interchange and intersection designs (divergent diamond interchanges, single-point urban interchanges, Michigan indirect left-turn intersections, and continuous flow intersections), and access management. These measures generally boost facility performance through the cost-effective addition of signal or lane capacity or both in unconventional ways to the facility.

More information on ATM measures and methods for their evaluation can be found in Chapter 35, Active Traffic Management.

4. EXAMPLE PROBLEMS

This part of the chapter describes the application of the automobile, pedestrian, bicycle, and transit methodologies through a series of example problems. Exhibit 16-15 provides an overview of these problems. The focus of the examples is to illustrate the multimodal facility evaluation process. An operational analysis level is used for all examples. The planning and preliminary engineering analysis level is identical to the operational analysis level in terms of the calculations except that default values are used when field-measured values are not available.

Exhibit 16-15
Example Problems

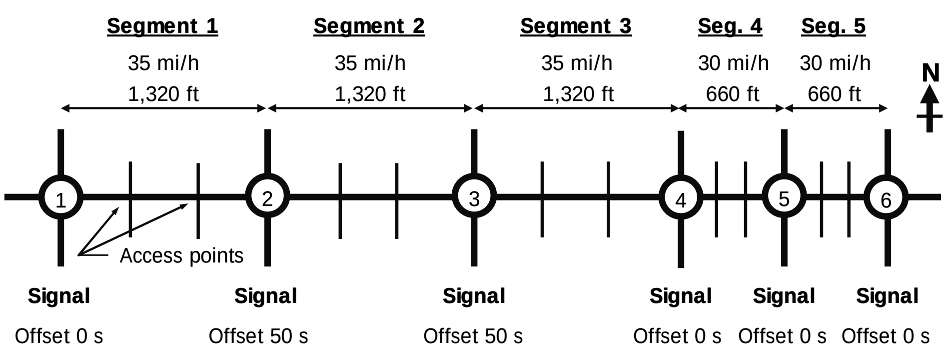
Problem Number	Description	Analysis Level
1	Auto-oriented urban street	Operational
2	Widen the sidewalks and add bicycle lanes on both sides of facility	Operational
3	Widen the sidewalks and add parking on both sides of facility	Operational

EXAMPLE PROBLEM 1: AUTO-ORIENTED URBAN STREET

The Urban Street Facility

A 1-mi urban street facility is shown in Exhibit 16-16. It is located in a downtown area and oriented in an east–west travel direction. The facility consists of five segments with a signalized boundary intersection for each segment. Segments 1, 2, and 3 are 1,320 ft long and have a speed limit of 35 mi/h. Segments 4 and 5 are 660 ft long and have a speed limit of 30 mi/h. Each segment has two access point intersections.

Exhibit 16-16
Example Problem 1: Urban Street Schematic

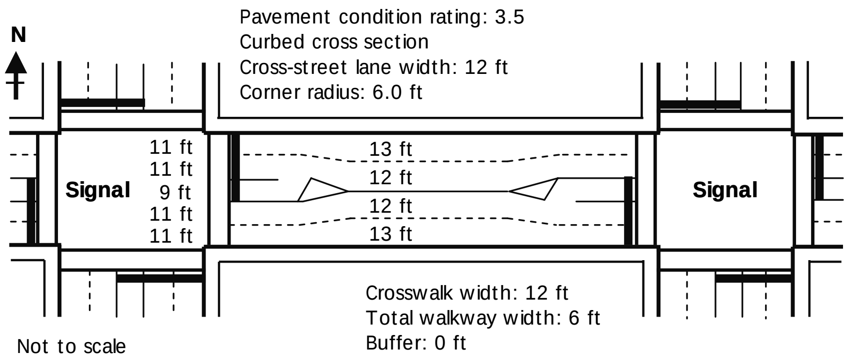


Segments 1, 2, and 3 pass through a mixture of office and strip commercial. Segments 4 and 5 are in a built-up shopping area.

The geometry of the typical street segment is shown in Exhibit 16-17. It is the same for each segment. The street has a curbed, four-lane cross section with two lanes in each direction. There is a 1.5-ft curb-and-gutter section on each side of the street. There are 200-ft left-turn bays on each approach to each signalized intersection. Right-turn vehicles share the outside lane with through vehicles on each intersection approach. A 6-ft sidewalk is provided on each side of the street adjacent to the curb. No fixed objects are located along the outside of the

sidewalk. Midsegment pedestrian crossings are legal. No bicycle lanes are provided on the facility or its cross streets. No parking is allowed along the street.

Exhibit 16-17
Example Problem 1: Segment Geometry



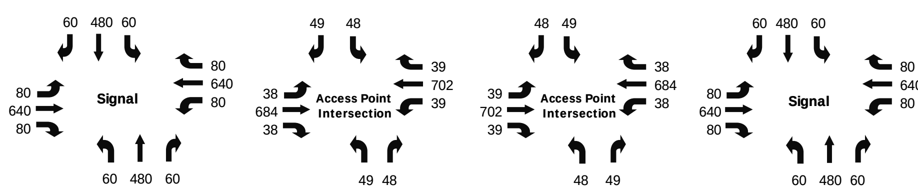
The Question

What are the travel speed and LOS of the automobile, pedestrian, bicycle, and transit modes in both directions of travel along the facility?

The Facts

The traffic counts for one segment are shown in Exhibit 16-18. The counts are the same for all of the other segments. The counts were taken during the 15-min analysis period of interest. However, they have been converted to hourly flow rates.

Exhibit 16-18
Example Problem 1: Intersection Turn Movement Counts



The signalization conditions are shown in Exhibit 16-19. The conditions shown are identified as belonging to Signalized Intersection 1; however, they are the same for the other signalized intersections (with exception of offset). The signals operate with coordinated–actuated control. The left-turn movements on the northbound and southbound approaches operate under permitted control. The left-turn movements on the major street operate as protected–permitted in a lead–lead sequence.

Exhibit 16-19 indicates that the passage time for each phase is 2.0 s. The minimum green setting is 5 s for the major-street left-turn phases and 18 s for the cross-street phases. The offset to Phase 2 (the reference phase) end-of-green interval is 0.0 s. The offset for each of the other intersections is shown in Exhibit 16-16. A fixed-force mode is used to ensure coordination is maintained. The cycle length is 100 s.

Geometric conditions and traffic characteristics for Signalized Intersection 1 are shown in Exhibit 16-20. They are the same for the other signalized

intersections. The movement numbers follow the numbering convention shown in Exhibit 18-2 of Chapter 18.

Exhibit 16-19
Example Problem 1: Signal Conditions for Intersection 1

Signalized Intersection 1								
General Information 19								
Cross Street:	First Avenue				**Analysis Period:**	AM peak		
Phase Sequence and Left-Turn Mode								
Major street sequence (movement numbers shown)	5 & 1 left leading				Cross street sequence (movement numbers shown)	No exclusive phase for 3 or 7		
Major street left-turn mode (movement numbers shown)	5/1 Protected+Permitted				Cross street left-turn mode (movement numbers shown)	3/7 Permitted		
Phase Settings								
Approach	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Phase number	5	2	1	6		8		4
Movement	L	T+R	L	T+R		L+T+R		L+T+R
Lead/lag left-turn phase	Lead	--	Lead	--		--		--
Left-turn mode	Pr/Pm	--	Pr/Pm	--		Perm.		Perm.
Passage time, s	2.0	--	2.0	--		2.0		2.0
Minimum green, s	5	--	5	--		18		18
Yellow + red clear, s	4.0	4.0	4.0	4.0		4.0		4.0
Phase split, s	20	45	20	45		35		35
Recall	No	--	No	--	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes
Ref. Phase: 2	Offset, s:	0	Offset Ref.:	End of Green	Force Mode:	Fixed		
	Cycle, s:	100						
Enable Simultaneous Gap-Out? Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑				Enable Dallas Left-Turn Phasing? Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐				

Exhibit 16-20
Example Problem 1: Geometric Conditions and Traffic Characteristics for Signalized Intersection 1

Signalized Intersection 1												
Signalized Intersection Input Data *(In each column, enter the volume and lanes data. For all other blue cells, enter values only if there is one or more lanes.)*												
Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
Lanes	1	2	0	1	2	0	1	2	0	1	2	0
Turn bay length, ft	200			200			200			200		
Sat. flow rate, veh/h/ln	1760	1829		1760	1829		1826	1838		1826	1838	
Platoon ratio	1.000	1.333		1.000	1.333		1.000	1.000		1.000	1.000	
Initial queue, veh	0	0		0	0		0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Stop line det. length, ft	40			40			40	40		40	40	
Max. allow. hdwy, s/veh	3.9			3.9			3.9	2.9		3.9	2.9	

The saturation flow rate is determined by using the procedure described in Chapter 18. All intersection movements include 3% heavy vehicles. The segment and intersection approaches are effectively level. No parking is allowed along the facility or its cross-street approaches. With a few exceptions (discussed below), local buses stop on the eastbound and westbound approaches to each signalized intersection at a rate of 3 buses/h.

Arrivals for all cross-street movements are effectively random, so a platoon ratio of 1.00 is used. The through movement arriving to the eastbound approach at Intersection 1 exhibits favorable progression from an upstream signal, so a platoon ratio of 1.33 is used. For similar reasons, a ratio of 1.33 is also used for the through movement arriving to the westbound approach at Intersection 6. Right-turn-on-red volume is estimated at 5.0% of the right-turn volume.

Each segment has a barrier curb along the outside of the street in each direction of travel. Allowing for the upstream signal width, the percentage of the segment length with curb is estimated at 94% for Segments 1, 2, and 3. It is estimated as 88% for Segments 4 and 5.

The traffic and lane assignment data for the two access point intersections for Segment 1 are shown in Exhibit 16-21. These data are the same for the other segments; however, the access point locations (shown in the first column) are

reduced by one-half for Segments 4 and 5. The movement numbers follow the numbering convention shown in Exhibit 19-3 of Chapter 19, Two-Way STOP-Controlled Intersections. There are no turn bays on the segment at the two access point intersections.

Exhibit 16-21
Example Problem 1: Access Point Data

Access Point Input Data													
Access	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Point	Movement	L	T	R	L	T	R	L	T	R	L	T	R
Location,ft	Movement number	1	2	3	4	5	6	7	8	9	10	11	12
440	Volume, veh/h	38	684	38	39	702	39	49	0	48	48	0	49
West end	Lanes	0	2	0	0	2	0	1	0	1	1	0	1
880	Volume, veh/h	39	702	39	38	684	38	48	0	49	49	0	48
	Lanes	0	2	0	0	2	0	1	0	1	1	0	1

A low wall is located along about 25% of the sidewalk in Segments 1, 2, and 3. In contrast, 10% of the sidewalk along Segments 4 and 5 is adjacent to a low wall, 35% to a building face, and 15% to a window display.

Office and strip commercial activity along Segments 1, 2, and 3 generates a pedestrian volume of 100 p/h on the adjacent sidewalks and crosswalks. Shopping activity along Segments 4 and 5 generates a pedestrian volume of 300 p/h on the adjacent sidewalks and crosswalks. A lack of bicycle lanes has discouraged bicycle traffic on the facility and its cross streets; however, a bicycle volume of 1.0 bicycle/h is entered for each intersection approach.

Local buses stop on the eastbound and westbound approaches to each signalized intersection, with the exception of Intersection 5. There are no stops on either approach to Intersection 5. However, transit stops are provided along the facility at 0.25-mi intervals, so the service is considered to be local. As a result, the westbound transit frequency on Segment 5 and the eastbound transit frequency on Segment 4 are considered to be the same as for the adjacent segments (i.e., 3 buses/h). The bus dwell time at each stop averages 20 s. Buses arrive within 5 min of their scheduled time about 75% of the time and have a load factor of 0.80 passengers/seat. Each bus stop has a bench but no shelter.

Outline of Solution

This section outlines the results of the facility evaluation. To complete this evaluation, the automobile, pedestrian, and bicycle methodologies in Chapter 18 were used to evaluate each of the signalized intersections on the facility. The procedure in Chapter 19 was used to estimate pedestrian delay when crossing at a midsegment location. The automobile, pedestrian, bicycle, and transit methodologies in Chapter 17 were then used to evaluate both directions of travel on each segment. Finally, the methodologies described in Section 2 were used to evaluate all four travel modes in both directions of travel on the facility. The findings from each evaluation are summarized in the following three subparts.

Intersection Evaluation

The results of the evaluation of Intersection 1 (i.e., First Avenue) are shown in Exhibit 16-22. The results for Intersections 2, 3, and eastbound Intersection 4 are similar. In contrast, Intersections 5 and 6 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Intersection 5 (i.e., Fifth Avenue) are shown in Exhibit 16-23. Intersection 6 and westbound Intersection 4 have similar results.

Exhibit 16-22
Example Problem 1: Intersection 1 Evaluation

Intersection	Approach	Eastbound			Westbound			Northbound			Southbound		
	Intersection Evaluation Summary												
First Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		100			100			100			100	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	2	0	1	2	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.17	0.34	0.34	0.15	0.34	0.34	0.36	0.62	0.62	0.36	0.62	0.62
20.4	Control delay, s/veh	7.78	5.75	5.77	7.04	13.38	13.73	43.18	34.24	34.28	43.18	34.24	34.28
Int. level of service	Stop rate, stops/veh	0.39	0.20	0.20	0.32	0.48	0.49	0.85	0.77	0.77	0.85	0.77	0.77
C	Level of service	**A**	**A**	**A**	**A**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	93.7			93.7			93.7			93.7		
	Crosswalk location	Crossing major			Crossing major			Crossing minor			Crossing minor		
	Crosswalk circulation area, ft2/p	75.9			75.9			82.4			82.4		
	Pedestrian delay, s/p	42.3			42.3			42.3			42.3		
	Pedestrian LOS score	2.75			2.75			2.66			2.66		
	Level of service	**C**			**C**			**B**			**B**		
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	n.a.			n.a.			n.a.			n.a.		
	Bicycle LOS score	3.72			3.72			2.87			2.87		
	Level of service	**D**			**D**			**C**			**C**		

Exhibit 16-23
Example Problem 1: Intersection 5 Evaluation

Intersection	Approach	Eastbound			Westbound			Northbound			Southbound		
	Intersection Evaluation Summary												
Fifth Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		300			300			300			300	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	2	0	1	2	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.16	0.34	0.34	0.16	0.34	0.34	0.36	0.62	0.62	0.36	0.62	0.62
20.2	Control delay, s/veh	7.73	8.74	8.31	7.63	9.84	9.48	43.10	34.13	34.17	43.10	34.13	34.17
Int. level of service	Stop rate, stops/veh	0.38	0.31	0.29	0.37	0.35	0.34	0.86	0.78	0.78	0.86	0.78	0.78
C	Level of service	**A**	**A**	**A**	**A**	**A**	**A**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	14.8			14.8			14.8			14.8		
	Crosswalk location	Crossing major			Crossing major			Crossing minor			Crossing minor		
	Crosswalk circulation area, ft2/p	24.5			24.5			26.7			26.7		
	Pedestrian delay, s/p	42.3			42.3			42.3			42.3		
	Pedestrian LOS score	2.70			2.70			2.62			2.62		
	Level of service	**B**			**B**			**B**			**B**		
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	n.a.			n.a.			n.a.			n.a.		
	Bicycle LOS score	3.72			3.72			2.87			2.87		
	Level of service	**D**			**D**			**C**			**C**		

Both exhibits indicate that the major-street vehicular through movements (i.e., eastbound Movement 2 and westbound Movement 6) operate with very low delay and few stops. The LOS is A and B for the eastbound and westbound through movements, respectively.

Pedestrian circulation area on the corners of Intersection 1 is generous, with pedestrians having the ability to move in their desired path without conflict. Corner circulation area at Intersection 5 is restricted, with pedestrians having very limited ability to pass slower pedestrians.

At Intersection 1, the low pedestrian volume results in generous crosswalk circulation area. Pedestrians rarely need to adjust their path to avoid conflicts. In contrast, the high pedestrian volume at Intersection 5 results in a constrained crosswalk circulation area. Pedestrians frequently adjust their path to avoid conflict. At each intersection, pedestrians experience an average wait of about 42 s at the corner to cross the street in any direction. This delay is lengthy, and some pedestrians may not comply with the signal indications. At Intersection 1, the pedestrian LOS is C for the major-street crossing and B for the minor-street crossing. At Intersection 5, the pedestrian LOS is B for the major-street and minor-street crossings.

Bicycle lanes are not provided at any intersection, so bicycle delay is not computed. The lack of a bicycle lane combined with a moderately high traffic volume results in a bicycle LOS D on the eastbound and westbound approaches of Intersection 1 and Intersection 5.

Segment Evaluation

The results of the evaluation of Segment 1 (i.e., First Avenue to Second Avenue) are shown in Exhibit 16-24. The results for Segments 2 and 3 are similar. In contrast, Segments 4 and 5 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Segment 5 (i.e., Fifth Avenue to Sixth Avenue) are shown in Exhibit 16-25. Segment 4 has similar results.

Exhibit 16-24
Example Problem 1:
Segment 1 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
First Avenue to Second Avenue	***Basic Description***		
	Speed limit, mi/h	35	35
	Vehicle volume, veh/h	800	800
	Through lanes, ln	2	2
Segment length, ft 1,320	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	40.3	40.3
	Travel speed, mi/h	23.8	23.2
	Spatial stop rate, stops/mi	1.77	1.92
	Level of service	**C**	**C**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	593.9	593.9
	Pedestrian travel speed, ft/s	3.54	3.54
	Pedestrian LOS score	3.76	3.76
	Level of service	**D**	**D**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.24	4.24
	Level of service	**D**	**D**
	Transit Level of Service		
	Transit travel speed, mi/h	12.7	12.5
	Transit LOS score	3.16	3.19
	Level of service	**C**	**C**

Exhibit 16-25
Example Problem 1:
Segment 5 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
Fifth Avenue to Sixth Avenue	***Basic Description***		
	Speed limit, mi/h	30	30
	Vehicle volume, veh/h	800	800
	Through lanes, ln	2	2
Segment length, ft 660	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	37.9	37.9
	Travel speed, mi/h	17.6	17.3
	Spatial stop rate, stops/mi	2.68	2.80
	Level of service	**D**	**D**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	153.3	153.3
	Pedestrian travel speed, ft/s	3.18	3.18
	Pedestrian LOS score	3.67	3.67
	Level of service	**D**	**D**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.48	4.48
	Level of service	**E**	**E**
	Transit Level of Service		
	Transit travel speed, mi/h	7.7	17.3
	Transit LOS score	3.64	2.79
	Level of service	**D**	**C**

Exhibit 16-24 indicates that the vehicular through movements on Segment 1 in the eastbound and westbound travel directions have a travel speed of 24 and 23 mi/h, respectively (i.e., about 58% of the base free-flow speed). The LOS of each movement is C. In contrast, Exhibit 16-25 indicates that the through movements have a travel speed of only about 17 mi/h on Segment 5 (or 46% of the base free-flow speed), which is LOS D. Vehicles stop at a rate of about 1.8 stops/mi on Segment 1 and about 2.7 stops/mi on Segment 5.

Pedestrian space on the sidewalk along the segment is generous on Segment 1 and adequate on Segment 5. These characterizations are based on

Exhibit 16-11 and an assumed dominance of platoon flow for Segments 4 and 5. Pedestrians on these sidewalks can walk freely without having to alter their path to accommodate other pedestrians. The segment travel speed (3.54 ft/s for Segment 1 and 3.18 ft/s for Segment 5) is adequate, but would desirably exceed 4.0 ft/s. Nevertheless, the sidewalk is near the traffic lanes and crossing the street at a midsegment location can be difficult. As a result, the pedestrian LOS is D on all segments.

Bicycle lanes are not provided along the segment, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a moderately high traffic volume results in a bicycle LOS D for both directions of travel on Segment 1. Bicycle service on Segment 5 is also poor. However, the short spacing between access points on Segment 5, relative to Segment 1, further degrades service quality such that the bicycle LOS on Segment 5 is E.

Transit travel speed is about 12 mi/h on Segment 1 and corresponds to LOS C. On Segment 5, the travel speed is about 8 mi/h and 17 mi/h in the eastbound and westbound directions, respectively. The low speed for the eastbound direction results in LOS D. The higher speed for the westbound direction is due to the lack of a westbound transit stop on Segment 5. It results in LOS C for this direction.

Facility Evaluation

The methodology described in Section 2 is used to compute the aggregate performance measures for each travel direction along the facility. The results are shown in Exhibit 16-26. This exhibit indicates that the vehicle travel speed is about 22 mi/h in each travel direction (or 56% of the base free-flow speed). An overall LOS C applies to both vehicular movements on the facility; however, it is noted that LOS D applies to Segments 4 and 5. Vehicles incur stops along the facility at a rate of about 1.9 stops/mi.

Exhibit 16-26
Example Problem 1: Facility Evaluation

	Facility Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
Facility length, ft 5,280	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	39.7	39.7
	Travel speed, mi/h	22.3	22.1
	Spatial stop rate, stops/mi	1.86	1.93
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	298.6	298.6
	Pedestrian travel speed, ft/s	3.4	3.4
	Pedestrian LOS score	3.73	3.74
	Level of service	**D**	**D**
	Poorest perf. segment LOS	D	D
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.30	4.30
	Level of service	**E**	**E**
	Poorest perf. segment LOS	E	E
	Transit Level of Service		
	Transit travel speed, mi/h	12.4	12.3
	Transit LOS score	3.15	3.16
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D

Pedestrian space on the sidewalk along the facility is generous. Pedestrians on the sidewalks can walk freely without having to alter their path to accommodate other pedestrians. The facility travel speed of about 3.4 ft/s is

adequate, but would desirably exceed 4.0 ft/s. Nevertheless, the sidewalk is near the traffic lanes and crossing the street at a midsegment location can be difficult. As a result, the pedestrian LOS is D for both directions of travel.

Bicycle lanes are not provided along the facility, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a moderately high traffic volume results in an overall bicycle LOS E for both directions of travel.

Transit travel speed is about 12 mi/h on the facility in each direction of travel. An overall LOS C is assigned to each direction. The lower speed on westbound Segment 4 and eastbound Segment 5 is noted to result in LOS D for those segments.

EXAMPLE PROBLEM 2: PEDESTRIAN AND BICYCLE IMPROVEMENTS

The Urban Street Facility

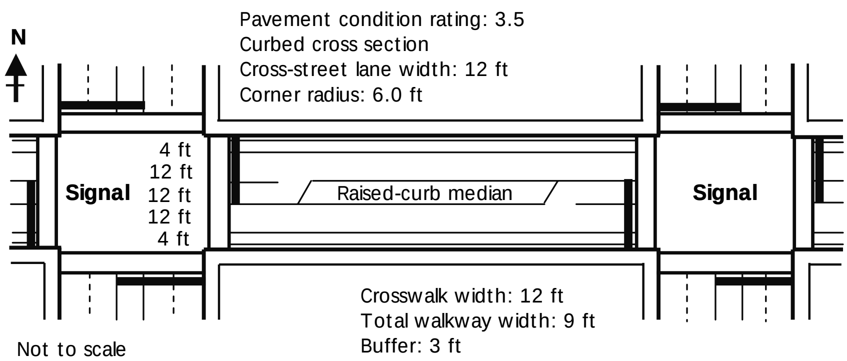
The 1-mi urban street facility shown in Exhibit 16-16 is being considered for geometric design modifications to improve pedestrian and bicycle service. The following changes to the facility are proposed:

- Eliminate one vehicle lane in each direction,
- Add a 12-ft raised-curb median,
- Add a 4-ft bicycle lane in each direction,
- Increase the total walkway width to 9 ft,
- Add a 3-ft buffer between the sidewalk and the curb, and
- Add bushes to the buffer using a 10-ft spacing.

No fixed objects are located along the outside of the sidewalk. The analysis for Example Problem 1 represents the existing condition, against which this alternative will be evaluated.

The geometry of the typical street segment is shown in Exhibit 16-27. It is the same for each segment. Additional segment details are provided in the discussion for Example Problem 1.

Exhibit 16-27
Example Problem 2:
Segment Geometry



The Question

What are the travel speed and LOS of the automobile, pedestrian, bicycle, and transit modes in both directions of travel along the facility?

The Facts

The traffic counts, signalization, and intersection geometry are listed in Exhibit 16-18 to Exhibit 16-21. They are unchanged from Example Problem 1.

Outline of Solution

This section outlines the results of the facility evaluation. To complete this evaluation, the automobile, pedestrian, and bicycle methodologies in Chapter 18 were used to evaluate each of the signalized intersections on the facility. The procedure in Chapter 19 was used to estimate pedestrian delay when crossing at a midsegment location. The automobile, pedestrian, bicycle, and transit methodologies in Chapter 17 were then used to evaluate both directions of travel on each segment. Finally, the methodologies described in Section 2 were used to evaluate all four travel modes in both directions of travel on the facility. The findings from each evaluation are summarized in the following three subparts.

Intersection Evaluation

The results of the evaluation of Intersection 1 (i.e., First Avenue) are shown in Exhibit 16-28. The results for Intersections 2, 3, and eastbound Intersection 4 are similar. In contrast, Intersections 5 and 6 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Intersection 5 (i.e., Fifth Avenue) are shown in Exhibit 16-29. Intersection 6 and westbound Intersection 4 have similar results.

Exhibit 16-28
Example Problem 2: Intersection 1 Evaluation

	Intersection Evaluation Summary												
Intersection	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
First Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		100			100			100			100	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	0	1	1	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.20	0.68	0.68	0.18	0.68	0.68	0.36	0.62	0.62	0.36	0.62	0.62
21.8	Control delay, s/veh	10.70	9.77	9.77	8.61	14.36	14.36	43.19	34.26	34.30	43.19	34.26	34.30
Int. level of service	Stop rate, stops/veh	0.54	0.24	0.24	0.45	0.46	0.46	0.85	0.77	0.77	0.85	0.77	0.77
C	Level of service	**B**	**A**	**A**	**A**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location		Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound	
	Corner circulation area, ft2/p		282.1			282.1			282.1			282.1	
	Crosswalk location			Crossing major			Crossing major			Crossing minor			Crossing minor
	Crosswalk circulation area, ft2/p			69.7			69.7			82.5			82.4
	Pedestrian delay, s/p			42.3			42.3			42.3			42.3
	Pedestrian LOS score			2.63			2.63			2.66			2.66
	Level of service			**B**			**B**			**B**			**B**
	Bicycle Level of Service												
	Bicycle delay, s/bicycle		8.3			8.3			n.a.			n.a.	
	Bicycle LOS score		2.99			2.99			2.77			2.77	
	Level of service		**C**			**C**			**C**			**C**	

Both exhibits indicate that the vehicular through movements on the facility (i.e., eastbound Movement 2 and westbound Movement 6) operate with low delay and few stops. For the eastbound through movement, the LOS is A at Intersection 1 and B at Intersection 5. The LOS is B for the westbound through movement at both intersections. Relative to Example Problem 1, the delay for the through movements has increased by a few seconds at Intersection 1 and by about 8 s at Intersection 5. This increase is sufficient to lower the LOS designation for the eastbound through movement at Intersection 5 (i.e., from A to B).

Exhibit 16-29
Example Problem 2:
Intersection 5 Evaluation

Intersection	Intersection Evaluation Summary												
Intersection	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Fifth Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		300			300			300			300	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	0	1	1	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.21	0.68	0.68	0.21	0.68	0.68	0.36	0.62	0.62	0.36	0.62	0.62
24.5	Control delay, s/veh	11.53	17.32	17.32	11.72	16.79	16.79	43.12	34.18	34.23	43.12	34.18	34.23
Int. level of service	Stop rate, stops/veh	0.59	0.58	0.58	0.59	0.56	0.56	0.86	0.78	0.78	0.86	0.78	0.78
C	Level of service	**B**	**B**	**B**	**B**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	77.6			77.6			77.6			77.6		
	Crosswalk location	Crossing major			Crossing major			Crossing minor			Crossing minor		
	Crosswalk circulation area, ft2/p	22.4			22.4			26.6			26.6		
	Pedestrian delay, s/p	42.3			42.3			42.3			42.3		
	Pedestrian LOS score	2.55			2.55			2.62			2.62		
	Level of service	**B**			**B**			**B**			**B**		
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	8.3			8.3			n.a.			n.a.		
	Bicycle LOS score	2.99			2.99			2.77			2.77		
	Level of service	**C**			**C**			**C**			**C**		

Pedestrian circulation area on the corners of Intersections 1 and 5 is generous, with few instances of conflict. This condition is greatly improved from Example Problem 1 and reflects the provision of wider sidewalks.

Relative to Example Problem 1, the reduction in through lanes has reduced the time provided to pedestrians to cross the major street. This reduction resulted in larger pedestrian groups using the crosswalk and a small reduction in crosswalk pedestrian space. At Intersection 1, pedestrian space is still generous, with few instances of conflict. At Intersection 5, the problem is amplified by a higher pedestrian demand. Pedestrian space in the crosswalks is constrained, and pedestrians are likely to find that their ability to pass slower pedestrians is limited.

At each intersection, pedestrians experience an average wait of about 42 s at the corner to cross the street in any direction. This condition has not changed from Example Problem 1.

At both intersections, the pedestrian LOS is B for the major-street and minor-street crossings. Relative to Example Problem 1, the pedestrian LOS score has improved by about the same amount at all intersections. At Intersection 1, this change is sufficient to result in a change in service level (i.e., from C to B).

Bicyclists using the bicycle lanes are expected to be delayed about 8 s/bicycle on both the eastbound and westbound approaches to each intersection. This level of delay is desirably low. However, the bicycle lane is relatively narrow at 4 ft, which leads to LOS C on the eastbound and westbound approaches of both intersections. This LOS is noted to be an improvement over the LOS D identified in Example Problem 1.

Segment Evaluation

The results of the evaluation of Segment 1 (i.e., First Avenue to Second Avenue) are shown in Exhibit 16-30. The results for Segments 2 and 3 are similar. In contrast, Segments 4 and 5 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from the other intersections. The results for Segment 5 (i.e., Fifth Avenue to Sixth Avenue) are shown in Exhibit 16-31. Segment 4 has similar results.

Exhibit 16-30
Example Problem 2: Segment 1 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
First Avenue to Second Avenue	***Basic Description***		
	Speed limit, mi/h	35	35
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 1,320	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	37.4	37.4
	Travel speed, mi/h	21.3	21.3
	Spatial stop rate, stops/mi	1.83	1.82
	Level of service	**C**	**C**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	809.9	809.9
	Pedestrian travel speed, ft/s	3.55	3.55
	Pedestrian LOS score	2.74	2.74
	Level of service	**B**	**B**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	13.18	13.18
	Bicycle LOS score	3.87	3.87
	Level of service	**D**	**D**
	Transit Level of Service		
	Transit travel speed, mi/h	10.3	10.4
	Transit LOS score	3.42	3.42
	Level of service	**C**	**C**

Exhibit 16-31
Example Problem 2: Segment 5 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
Fifth Avenue to Sixth Avenue	***Basic Description***		
	Speed limit, mi/h	30	30
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 660	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	35.3	35.3
	Travel speed, mi/h	12.7	13.2
	Spatial stop rate, stops/mi	4.74	4.47
	Level of service	**E**	**E**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	225.4	225.4
	Pedestrian travel speed, ft/s	3.18	3.18
	Pedestrian LOS score	2.72	2.72
	Level of service	**B**	**B**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	11.75	11.75
	Bicycle LOS score	4.10	4.10
	Level of service	**D**	**D**
	Transit Level of Service		
	Transit travel speed, mi/h	5.2	13.2
	Transit LOS score	4.00	3.14
	Level of service	**D**	**C**

Exhibit 16-30 indicates that the vehicular through movements on Segment 1 in the eastbound and westbound travel directions have a travel speed of about 21 mi/h (i.e., about 57% of the base free-flow speed). LOS C applies to both movements. In contrast, Exhibit 16-31 indicates that the through movements have a travel speed of only about 13 mi/h on Segment 5 (or 37% of the base free-flow speed), which is LOS E. Vehicles stop at a rate of about 1.8 stops/mi on Segment 1 and about 4.6 stops/mi on Segment 5. Relative to Example Problem 1, conditions have notably degraded for vehicles traveling along Segment 5.

Pedestrian space on the sidewalk along the segment is generous on Segment 1. Pedestrians can walk freely without having to alter their path to accommodate other pedestrians. Pedestrian space is adequate on Segment 5, with pedestrians in platoons occasionally needing to adjust their path to avoid conflict. These characterizations are based on Exhibit 16-11 and an assumed dominance of platoon flow for Segments 4 and 5. Relative to Example Problem 1, the sidewalks are more distant from the traffic lanes and crossing the street at a midsegment location is easier because of the raised curb median. As a result, the pedestrian LOS is B on all segments.

Bicyclists using the bicycle lanes experience a travel speed of 13 mi/h on Segment 1 and 12 mi/h on Segment 5. This travel speed is considered desirable. However, the bicycle lane is relatively narrow at 4 ft, so a bicycle LOS D results for both directions of travel on each segment. While still poor, the bicycle LOS scores indicate that bicycle service has improved on both segments relative to that found in Example Problem 1. In fact, the bicycle LOS on Segment 5 has improved by one letter designation.

Transit travel speed is 10 mi/h on Segment 1 and corresponds to LOS C. On Segment 5, the travel speed is about 5 mi/h and 13 mi/h in the eastbound and westbound directions, respectively. The low speed for the eastbound direction results in LOS D. The higher speed for the westbound direction is due to the lack of a westbound transit stop on Segment 5. It results in LOS C. Relative to Example Problem 1, the slower vehicular travel speed has increased the transit LOS scores, but not enough to change the designated service level.

Facility Evaluation

The methodology described in Section 2 is used to compute the aggregate performance measures for each travel direction along the facility. The results are shown in Exhibit 16-32. This exhibit indicates that the vehicle travel speed is about 18 mi/h in each travel direction (or 49% of the base free-flow speed). An overall LOS D applies to vehicle travel in each direction on the facility. It is noted that LOS E applies to Segments 4 and 5. Vehicles incur stops along the facility at a rate of about 2.6 stops/mi. Relative to Example Problem 1, vehicular travel speed has dropped about 4 mi/h, and LOS has degraded one level for this scenario.

Exhibit 16-32
Example Problem 2: Facility Evaluation

	Facility Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
	Vehicle Level of Service		
	Base free-flow speed, mi/h	36.8	36.8
	Travel speed, mi/h	18.0	18.1
	Spatial stop rate, stops/mi	2.64	2.62
Facility length, ft	Level of service	**D**	**D**
5,280	Poorest perf. segment LOS	E	E
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	422.2	422.2
	Pedestrian travel speed, ft/s	3.4	3.4
	Pedestrian LOS score	2.74	2.74
	Level of service	**B**	**B**
	Poorest perf. segment LOS	B	B
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	12.8	12.8
	Bicycle LOS score	3.93	3.93
	Level of service	**D**	**D**
	Poorest perf. segment LOS	D	D
	Transit Level of Service		
	Transit travel speed, mi/h	9.3	9.3
	Transit LOS score	3.48	3.48
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D

Pedestrian space on the sidewalk along the facility is generous. Pedestrians on the sidewalks can walk freely without having to alter their path to accommodate other pedestrians. Increasing the separation between the sidewalk and traffic lanes and improving pedestrians' ability to cross the street at midsegment locations (by adding a raised-curb median) have resulted in an

overall pedestrian LOS B for both directions of travel. This level compares favorably with the LOS D indicated in Example Problem 1.

Bicyclists in the bicycle lanes are estimated to experience an average travel speed of about 13 mi/h. This travel speed is considered desirable. However, the 4-ft bicycle lane is relatively narrow and produces LOS D. This level is one level improved over that found for Example Problem 1.

Transit travel speed is about 9 mi/h on the facility in each direction of travel. An overall LOS C is assigned to each direction. Relative to Example Problem 1, the LOS designation is unchanged; however, the transit speed is slower, and the transit LOS score indicates a small reduction in service.

EXAMPLE PROBLEM 3: PEDESTRIAN AND PARKING IMPROVEMENTS

The Urban Street Facility

The 1-mi urban street facility shown in Exhibit 16-16 is being considered for geometric design modifications to improve parking and pedestrian service. The following changes to the facility are proposed:

- Eliminate one vehicle lane in each direction,
- Add a 12-ft raised-curb median,
- Add a 9.5-ft parking lane in each direction, and
- Increase the total walkway width to 7 ft.

No fixed objects will be located along the outside of the sidewalk. The on-street parking is expected to be occupied 50% of the time. Parking maneuvers are estimated to cause 1.8 s/veh additional delay on Segments 1, 2, and 3. On Segments 4 and 5, these maneuvers are estimated to cause 0.3 s/veh additional delay. The analysis for Example Problem 1 represents the existing condition, against which this alternative will be evaluated.

The geometry of the typical street segment is shown in Exhibit 16-33. It is the same for each segment. Additional segment details are provided in the discussion for Example Problem 1.

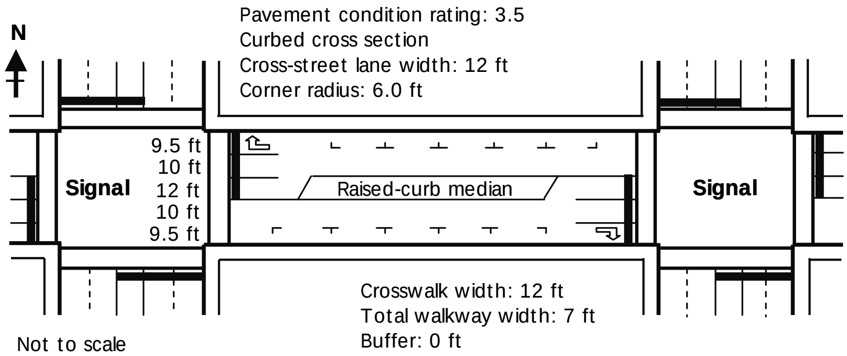
The Question

What are the travel speed and LOS of the automobile, pedestrian, bicycle, and transit modes in both directions of travel along the facility?

The Facts

The traffic counts, signalization, and intersection geometry are listed in Exhibit 16-18 to Exhibit 16-21. They are unchanged from Example Problem 1.

Exhibit 16-33
Example Problem 3:
Segment Geometry



Outline of Solution

This section outlines the results of the facility evaluation. To complete this evaluation, the automobile, pedestrian, and bicycle methodologies in Chapter 18 were used to evaluate each of the signalized intersections on the facility. The procedure in Chapter 19 was used to estimate pedestrian delay when crossing at a midsegment location. The automobile, pedestrian, bicycle, and transit methodologies in Chapter 17 were then used to evaluate both directions of travel on each segment. Finally, the methodologies described in Section 2 were used to evaluate all four travel modes in both directions of travel on the facility. The findings from each evaluation are summarized in the following three subparts.

Intersection Evaluation

The results of the evaluation of Intersection 1 (i.e., First Avenue) are shown in Exhibit 16-34. The results for Intersections 2, 3, and eastbound Intersection 4 are similar. In contrast, Intersections 5 and 6 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Intersection 5 (i.e., Fifth Avenue) are shown in Exhibit 16-35. Intersection 6 and westbound Intersection 4 have similar results.

Exhibit 16-34
Example Problem 3:
Intersection 1 Evaluation

	Intersection Evaluation Summary												
Intersection	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
First Avenue	***Basic Description***												
	Applicable lane assignments	L	T	R	L	T	R	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		100			100			100			100	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	1	1	1	1	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.19	0.59	0.09	0.16	0.59	0.09	0.36	0.62	0.62	0.36	0.62	0.62
21.8	Control delay, s/veh	10.01	7.99	9.00	7.40	16.60	11.49	43.19	34.26	34.30	43.19	34.26	34.30
Int. level of service	Stop rate, stops/veh	0.51	0.22	0.34	0.38	0.56	0.42	0.85	0.77	0.77	0.85	0.77	0.77
C	Level of service	**B**	**A**	**A**	**A**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p		148.1			148.1			148.1			148.1	
	Crosswalk location			Crossing major			Crossing major			Crossing minor			Crossing minor
	Crosswalk circulation area, ft2/p			74.0			74.0			82.6			82.4
	Pedestrian delay, s/p			42.3			42.3			42.3			42.3
	Pedestrian LOS score			2.67			2.67			2.66			2.66
	Level of service			**B**			**B**			**B**			**B**
	Bicycle Level of Service												
	Bicycle delay, s/bicycle		n.a.			n.a.			n.a.			n.a.	
	Bicycle LOS score		4.27			4.27			2.83			2.83	
	Level of service		**E**			**E**			**C**			**C**	

Both exhibits indicate that the vehicular through movements on the facility (i.e., eastbound Movement 2 and westbound Movement 6) operate with very low delay and few stops. For the eastbound through movement, the LOS is A at Intersection 1 and B at Intersection 5. The LOS is B for the westbound through movement at both intersections. Relative to Example Problem 1, the delay for the

through movements has increased by a few seconds at both intersections. However, this increase is sufficient to lower the LOS designation for only the eastbound through movement at Intersection 5.

Exhibit 16-35
Example Problem 3: Intersection 5 Evaluation

Intersection	Intersection Evaluation Summary												
Intersection	Approach	Eastbound			Westbound			Northbound			Southbound		
Fifth Avenue	***Basic Description***												
	Applicable lane assignments	L	T	R	L	T	R	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		300			300			300			300	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	1	1	1	1	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.18	0.59	0.10	0.17	0.59	0.10	0.36	0.62	0.62	0.36	0.62	0.62
21.6	Control delay, s/veh	9.60	11.83	4.74	9.25	13.32	6.16	43.12	34.18	34.22	43.12	34.18	34.22
Int. level of service	Stop rate, stops/veh	0.51	0.39	0.19	0.49	0.45	0.24	0.86	0.78	0.78	0.86	0.78	0.78
C	Level of service	**A**	**B**	**A**	**A**	**B**	**A**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	33.0			33.0			33.0			33.0		
	Crosswalk location	Crossing major			Crossing major			Crossing minor			Crossing minor		
	Crosswalk circulation area, ft2/p	23.8			23.8			26.6			26.6		
	Pedestrian delay, s/p	42.3			42.3			42.3			42.3		
	Pedestrian LOS score	2.61			2.61			2.62			2.62		
	Level of service	**B**			**B**			**B**			**B**		
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	n.a.			n.a.			n.a.			n.a.		
	Bicycle LOS score	4.27			4.27			2.83			2.83		
	Level of service	**E**			**E**			**C**			**C**		

Pedestrian circulation area on the corners of Intersection 1 is generous. However, corner circulation area at Intersection 5 is constrained, with pedestrians frequently needing to adjust their path to avoid slower pedestrians. Regardless, this condition is improved from Example Problem 1 and reflects the provision of wider sidewalks.

Relative to Example Problem 1, the reduction in lanes has reduced the time provided to pedestrians to cross the major street. This reduction resulted in larger pedestrian groups using the crosswalk and a slight reduction in crosswalk pedestrian space. At Intersection 1, pedestrian space is generous. However, pedestrian space is constrained at Intersection 5, with pedestrians having limited ability to pass slower pedestrians as they cross the street.

At each intersection, pedestrians experience an average wait of about 42 s at the corner to cross the street in any direction. At both intersections, the pedestrian LOS is B for the major-street crossing and the minor-street crossing. The LOS designation has improved at Intersection 1 by one letter, relative to Example Problem 1, and remains unchanged at Intersection 5.

Bicycle lanes are not provided at any intersection, so bicycle delay is not computed. The lack of a bicycle lane combined with a high traffic volume results in a bicycle LOS E on the eastbound and westbound approaches of Intersection 1 and Intersection 5. This level is noted to be worse than the LOS D identified in Example Problem 1 because the traffic volume per lane has doubled.

Segment Evaluation

The results of the evaluation of Segment 1 (i.e., First Avenue to Second Avenue) are shown in Exhibit 16-36. The results for Segments 2 and 3 are similar. In contrast, Segments 4 and 5 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Segment 5 (i.e., Fifth Avenue to Sixth Avenue) are shown in Exhibit 16-37. Segment 4 has similar results.

Exhibit 16-36
Example Problem 3:
Segment 1 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	Eastbound	Westbound
First Avenue to Second Avenue	***Basic Description***		
	Speed limit, mi/h	35	35
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 1,320	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	37.4	37.4
	Travel speed, mi/h	20.0	19.5
	Spatial stop rate, stops/mi	2.05	2.22
	Level of service	**C**	**C**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	737.9	737.9
	Pedestrian travel speed, ft/s	3.55	3.55
	Pedestrian LOS score	2.89	2.89
	Level of service	**C**	**C**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.70	4.70
	Level of service	**E**	**E**
	Transit Level of Service		
	Transit travel speed, mi/h	10.5	10.2
	Transit LOS score	3.38	3.41
	Level of service	**C**	**C**

Exhibit 16-37
Example Problem 3:
Segment 5 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	Eastbound	Westbound
Fifth Avenue to Sixth Avenue	***Basic Description***		
	Speed limit, mi/h	30	30
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 660	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	35.3	35.3
	Travel speed, mi/h	14.9	14.5
	Spatial stop rate, stops/mi	3.35	3.59
	Level of service	**D**	**D**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	201.4	201.4
	Pedestrian travel speed, ft/s	3.18	3.18
	Pedestrian LOS score	2.87	2.87
	Level of service	**C**	**C**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.94	4.94
	Level of service	**E**	**E**
	Transit Level of Service		
	Transit travel speed, mi/h	6.2	14.5
	Transit LOS score	3.84	3.01
	Level of service	**D**	**C**

Exhibit 16-36 indicates that the vehicular through movements on Segment 1 in the eastbound and westbound travel directions have a travel speed of about 20 mi/h (i.e., about 53% of the base free-flow speed). LOS C applies to both movements. In contrast, Exhibit 16-37 indicates that the through movements have a travel speed of only about 15 mi/h on Segment 5 (or 42% of the base free-flow speed), which is LOS D. Vehicles stop at a rate of about 2.1 stops/mi on Segment 1 and about 3.5 stops/mi on Segment 5. Relative to Example Problem 1, conditions have degraded for vehicles traveling along these segments, but not enough to drop the LOS designation.

Pedestrian space on the sidewalk along the segment is generous on Segment 1 and adequate on Segment 5. These characterizations are based on Exhibit 16-11 and an assumed dominance of platoon flow for Segments 4 and 5. Pedestrians on these sidewalks can walk freely without having to alter their path to accommodate other pedestrians. Relative to Example Problem 1, the sidewalks are more distant from the traffic lanes, and crossing the street at a midsegment location is easier because of the raised-curb median. As a result, the pedestrian LOS is improved on all segments (i.e., from LOS D to C).

Bicycle lanes are not provided along the segment, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a high traffic volume results in a bicycle LOS E for both directions of travel on Segment 1 and Segment 5. Relative to Example Problem 1, conditions have degraded for bicyclists on all segments, and the LOS for Segment 1 has dropped by one level. This reduction in service is due largely to the increased density of vehicles in the mixed traffic lanes.

Transit travel speed is about 10 mi/h on Segment 1 and corresponds to LOS C. On Segment 5, the travel speed is about 6 mi/h and 14 mi/h in the eastbound and westbound directions, respectively. The low speed for the eastbound direction results in LOS D. The higher speed for the westbound direction is due to the lack of a westbound transit stop on Segment 5. It results in LOS C. Relative to Example Problem 1 the slower travel speed has increased the transit LOS scores, but not enough to change the designated service level.

Facility Evaluation

The methodology described in Section 2 is used to compute the aggregate performance measures for each travel direction along the facility. The results are shown in Exhibit 16-38. This exhibit indicates that the vehicle travel speed is about 19 mi/h in each travel direction (or 50% of the base free-flow speed). An overall LOS C applies to both vehicular movements on the facility; however, it is noted that LOS D applies to Segments 4 and 5. Vehicles incur stops along the facility at a rate of about 2.3 stops/mi. Relative to Example Problem 1, vehicular LOS has degraded, but not enough to drop the LOS designation.

Exhibit 16-38
Example Problem 3: Facility Evaluation

	Facility Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
	Vehicle Level of Service		
	Base free-flow speed, mi/h	36.8	36.8
	Travel speed, mi/h	18.7	18.5
	Spatial stop rate, stops/mi	2.23	2.33
Facility length, ft	Level of service	**C**	**C**
5,280	Poorest perf. segment LOS	D	D
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	381.1	381.1
	Pedestrian travel speed, ft/s	3.4	3.4
	Pedestrian LOS score	2.88	2.88
	Level of service	**C**	**C**
	Poorest perf. segment LOS	C	C
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.76	4.76
	Level of service	**E**	**E**
	Poorest perf. segment LOS	E	E
	Transit Level of Service		
	Transit travel speed, mi/h	10.2	10.1
	Transit LOS score	3.37	3.38
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D

Pedestrian space on the sidewalk along the facility is generous. Pedestrians on the sidewalks can walk freely without having to alter their path to accommodate other pedestrians. Increasing the separation between the sidewalk and traffic lanes and improving pedestrians' ability to cross the street (by adding a raised-curb median) result in an overall pedestrian LOS C for both directions of travel. This level compares favorably with the LOS D indicated in Example Problem 1.

Bicycle lanes are not provided along the facility, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a high traffic volume results in an overall bicycle LOS E for both directions of travel. Conditions have degraded slightly, relative to Example Problem 1, but not enough to drop the LOS designation.

Transit travel speed is about 10 mi/h on the facility in each direction of travel. An overall LOS C is assigned to each direction. Conditions have degraded slightly, relative to Example Problem 1, but not enough to drop the transit LOS designation.

5. REFERENCES

These references are available in the Technical Reference Library in Volume 4.

1. Bonneson, J. A., M. P. Pratt, and M. A. Vandehey. *Predicting the Performance of Automobile Traffic on Urban Streets: Final Report.* National Cooperative Highway Research Program Project 3-79. Texas Transportation Institute, Texas A&M University, College Station, Jan. 2008.
2. Dowling, R. G., D. B. Reinke, A. Flannery, P. Ryus, M. Vandehey, T. A. Petritsch, B. W. Landis, N. M. Rouphail, and J. A. Bonneson. *NCHRP Report 616: Multimodal Level of Service Analysis for Urban Streets.* Transportation Research Board of the National Academies, Washington, D.C., 2008.
3. Kittelson & Associates, Inc.; KFH Group, Inc.; Parsons Brinckerhoff Quade and Douglass, Inc.; and K. Hunter-Zaworski. *TCRP Report 100: Transit Capacity and Quality of Service Manual*, 2nd ed. Transportation Research Board of the National Academies, Washington, D.C., 2003.

CHAPTER 17
URBAN STREET SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

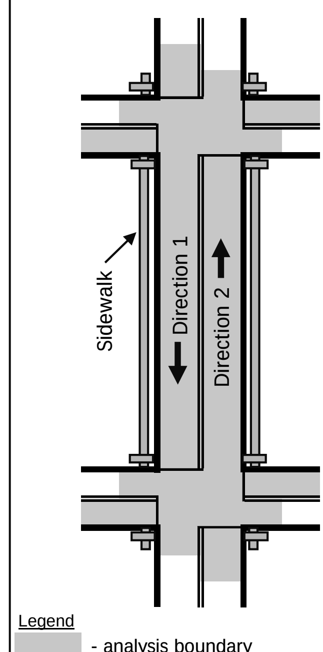
Chapter 17, Urban Street Segments, describes a methodology for evaluating the capacity and quality of service provided to road users traveling along an urban street segment. However, the methodology is much more than just a tool for evaluating capacity and quality of service. The methodology includes an array of performance measures that more fully describes segment operation for multiple travel modes. These measures serve as clues in identifying the source of problems and provide insight into the development of effective improvement strategies. The analyst is encouraged to consider the full range of measures when using this methodology.

OVERVIEW OF THE METHODOLOGY

This chapter's methodology is applicable to an urban or suburban street segment. The segment can be part of an arterial or collector street with one-way or two-way vehicular traffic flow. The intersections on the segment can be signalized or unsignalized.

Analysis Boundaries

The segment analysis boundary is defined by the roadway right-of-way and the operational influence area of each boundary intersection. The influence area of a boundary intersection extends backward from the intersection on each intersection leg. The size of this area is leg-specific and includes the most distant extent of any intersection-related queue expected to occur during the study period. For these reasons, the analysis boundaries should be established for each intersection on the basis of the conditions present during the analysis period. Practically speaking, the influence area should extend at least 250 ft back from the stop line on each intersection leg.



Analysis Level

Analysis level describes the level of detail used in applying the methodology. Three levels are recognized:

- Operational,
- Design, and
- Planning and preliminary engineering.

The operational analysis is the most detailed application and requires the most information about the traffic, geometric, and signalization conditions. The design analysis also requires detailed information about the traffic conditions and the desired level of service (LOS) as well as information about either the geometric or signalization conditions. The design analysis then seeks to determine reasonable values for the conditions not provided. The planning and preliminary engineering analysis requires only the most fundamental types of information from the analyst. Default values are then used as substitutes for other input data. The subject of analysis level is discussed in more detail in the Applications section of this chapter.

Study Period and Analysis Period

The study period is the time interval represented by the performance evaluation. It consists of one or more consecutive analysis periods. An analysis period is the time interval evaluated by a single application of the methodology.

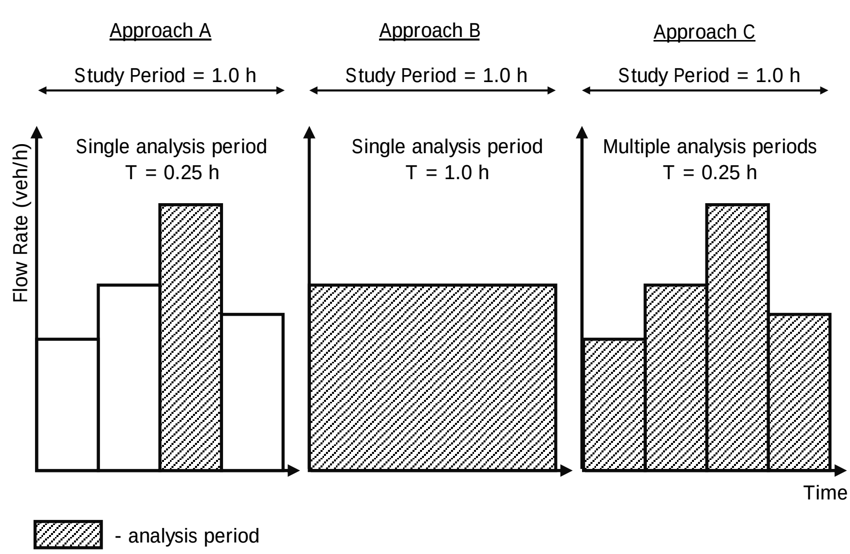
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the duration of the analysis period is in the range of 0.25 to 1 h. The longer durations in this range are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1 h because traffic conditions are not typically steady for long time periods and because the adverse impact of short peaks in traffic demand may not be detected in the evaluation.

If an analysis period of interest has a demand volume that exceeds capacity, then the study period should include an initial analysis period with no initial queue and a final analysis period with no residual queue. This approach provides a more accurate estimate of the delay associated with the congestion.

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be separately reported. In this situation, reporting an average performance for the study period is not encouraged because it may obscure extreme values and suggest acceptable operation when in reality some analysis periods have unacceptable operation.

Exhibit 17-1 demonstrates three alternative approaches an analyst might use for a given evaluation. Note that other alternatives exist and that the study period can exceed 1 h. Approach A is the one that has traditionally been used and, unless otherwise justified, is the one that is recommended for use.

Exhibit 17-1
Three Alternative Study Approaches



Approach A is based on the evaluation of the peak 15-min period during the study period. The analysis period T is 0.25 h. The equivalent hourly flow rate used for the analysis is based on either a peak 15-min traffic count multiplied by four or a 1-h demand volume divided by the peak hour factor. The former option is preferred whenever traffic counts are available. The peak hour factor equals

the hourly count of vehicles divided by four times the peak 15-min count for a common hour interval. It is provided by the analyst or operating agency.

Approach B is based on the evaluation of one 1-h analysis period that is coincident with the study period. The analysis period T is 1.0 h. The flow rate used is equivalent to the 1-h demand volume (i.e., the peak hour factor is not used). This approach implicitly assumes that the arrival rate of vehicles is constant throughout the period of study. Therefore, the effects of peaking within the hour may not be identified, and the analyst risks underestimating the delay actually incurred.

Approach C uses a 1-h study period and divides it into four 0.25-h analysis periods. This approach accounts for systematic flow rate variation among analysis periods. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay.

Performance Measures

A street segment's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road-user group. Performance measures cited in this chapter include automobile travel speed, automobile stop rate, automobile traveler perception score, pedestrian travel speed, pedestrian space, pedestrian perception score, bicycle travel speed, bicycle perception score, transit vehicle travel speed, transit wait–ride score, and transit passenger perception score.

LOS is also considered a performance measure. It is computed for the automobile, pedestrian, bicycle, and transit travel modes. It is useful for describing segment performance to elected officials, policy makers, administrators, or the public. LOS is based on one or more of the performance measures listed in the previous paragraph.

Travel Modes

This chapter describes a separate methodology for evaluating urban street performance from the perspective of motorists, pedestrians, bicyclists, or transit passengers. These methodologies are referred to as the automobile methodology, pedestrian methodology, bicycle methodology, and transit methodology.

Each methodology consists of a set of procedures for computing the quality of service provided to one mode. Collectively, they can be used to evaluate the urban street segment operation from a multimodal perspective.

Each methodology is focused on the evaluation of a street segment (with consideration given to the intersections that bound it). The aggregation of segment performance measures to obtain an estimate of facility performance is described in Chapter 16, Urban Street Facilities. Methodologies for evaluating the intersections on the urban street are described in Chapters 18 to 22.

The four methodologies described in this chapter are based largely on the products of two National Cooperative Highway Research Program projects (*1*, *2*). Contributions to the methodology from other research are referenced in the relevant sections.

The transit methodology described in this chapter is applicable to the evaluation of passenger service provided by local public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. Nonlocal transit vehicle speed and delay are evaluated by using the automobile methodology.

The phrase *automobile mode*, as used in this chapter, refers to travel by all motorized vehicles that can legally operate on the street, with the exception of local transit vehicles that stop to pick up passengers along the street. Unless explicitly stated otherwise, the word *vehicles* refers to motorized vehicles and includes a mixed stream of automobiles, motorcycles, trucks, and buses.

Lane Groups and Movement Groups

Lane group and *movement group* are phrases used to define combinations of intersection movements for the purpose of evaluating signalized intersection operation. These two terms are used extensively in Chapter 18, Signalized Intersections. They are also used in this chapter when the boundary intersection is signalized.

The automobile methodology in Chapter 18 is designed to evaluate the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group.* In general, a separate lane group is established for (*a*) each lane (or combination of adjacent lanes) that exclusively serves one movement and (*b*) each lane shared by two or more movements.

The concept of *movement groups* is also established to facilitate data entry. A separate movement group is established for (*a*) each turn movement with one or more exclusive turn lanes and (*b*) the through movement (inclusive of any turn movements that share a lane).

URBAN STREET SEGMENT DEFINED

For the purpose of analysis, the roadway is separated into individual elements that are physically adjacent and operate as a single entity in serving travelers. Two elements are commonly found on an urban street system: points and links. A *point* represents the boundary between links and is represented by an intersection or ramp terminal. A *link* represents a length of roadway between two points. A link and its boundary points are referred to as a *segment.*

Previous editions of this manual have allowed the evaluation of one direction of travel along a segment (even when it served two-way traffic). This approach is retained in this chapter for the analysis of bicycle and transit performance. For the analysis of pedestrian performance, this approach translates into the evaluation of sidewalk and street conditions on one side of the segment.

For the automobile methodology, a segment evaluation considers both directions of travel (when the street serves two-way traffic).

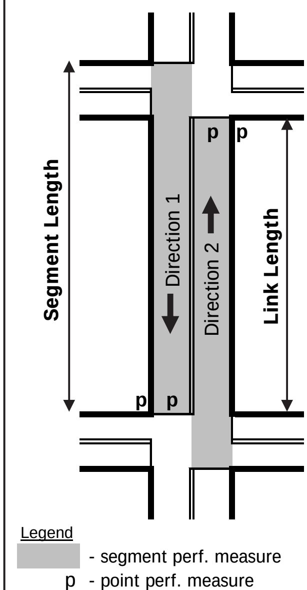
For the analysis of automobile performance, an analysis of only one travel direction (when the street serves two-way traffic) does not adequately recognize the interactions between vehicles at the boundary intersections and their influence on segment operation. For example, the automobile methodology in this edition of the *Highway Capacity Manual* (HCM) explicitly models the platoon

formed by the signal at one end of the segment and its influence on the operation of the signal at the other end of the segment. For these reasons, it is important to evaluate both travel directions on a two-way segment.

Points and Segments

The link and its boundary points must be evaluated together to provide an accurate indication of overall segment performance. For a given direction of travel along the segment, link and downstream point performance measures are combined to determine overall segment performance.

A segment performance measure combines link performance and point performance.



If the subject segment is within a coordinated signal system, then the following rules apply when the segment boundaries are identified:

- A signalized intersection (or ramp terminal) is always used to define a segment boundary.
- Only intersections (or ramp terminals) at which the segment through movement is uncontrolled (e.g., a two-way STOP-controlled intersection) can exist along the segment between the boundaries.

If the subject segment is not within a coordinated signal system, then the following rules apply when the segment boundaries are identified:

- An intersection (or ramp terminal) having a type of control that can impose on the segment through movement a legal requirement to stop or yield must always be used to define a segment boundary.
- An intersection (or ramp terminal) at which the segment through movement is uncontrolled (e.g., a two-way STOP-controlled intersection) may be used to define a segment boundary, but it is typically not done.

A midsegment traffic control signal provided for the exclusive use of pedestrians should not be used to define a segment boundary. This restriction reflects the fact that the methodologies described here were derived for, and calibrated with data from, street segments bounded by an intersection.

An access point intersection is an unsignalized intersection with one or two access point approaches to the segment. The approach can be a driveway or a public street. The through movements on the segment are uncontrolled at an access point intersection.

Segment Length Considerations

When a segment has a "short" length, then the interaction between traffic movements and traffic control devices at the two boundary intersections is sufficiently complex that a separate analysis of each element will not provide an accurate indication of urban street performance. This complication can occur regardless of the type of control present at the two boundary intersections; however, it is particularly complicated when the two intersections are signalized. The automobile methodology described in this chapter is not appropriate for the analysis of short segments. In contrast, the methodology described in Chapter 22, Interchange Ramp Terminals, is appropriate for the analysis of short segments at signalized interchanges.

It is difficult to define specific conditions under which a segment is short. However, two general rules apply in making this determination. First, a segment is considered to be short if the queue frequently extends back from one intersection into the other intersection (i.e., spills back) during the analysis period. Second, a segment is considered to be short if the through signal phase duration at the downstream intersection is longer than that needed to serve all the vehicles that store on the segment plus any vehicles that can enter it from the upstream signalized intersection while the downstream phase is green. This situation results in "demand starvation." It leads to the inefficient use of the downstream through phase and the retention of unserved vehicles on the approaches to the upstream intersection. In general, segments that are bounded by signalized intersections and are shorter than 400 ft may experience one or both of these conditions.

Platoons formed at a signalized intersection are typically dispersed by the time they reach a point about 0.6 mi downstream of the signal. This distance can vary depending on the amount of access point activity along the street and the speed of the traffic stream. Regardless, the influence of platoons on urban street operation is very likely to be negligible when segment length exceeds 2 mi. Therefore, if a segment exceeds 2 mi in length and its boundary points are signalized, then the analyst should evaluate the segment as an uninterrupted-flow highway segment with isolated intersections.

LOS CRITERIA

This subsection describes the LOS criteria for the automobile, pedestrian, bicycle, and transit modes. The criteria for the automobile mode are different from the criteria used for the nonautomobile modes. Specifically, the automobile mode criteria are based on performance measures that are field-measurable and perceivable by travelers. The criteria for the pedestrian and bicycle modes are based on scores reported by travelers indicating their perception of service quality. The criteria for the transit mode are based on measured changes in transit patronage due to changes in service quality.

Automobile Mode

All uses of the word "volume" or the phrase "volume-to-capacity ratio" in this chapter refer to demand volume or demand-volume-to-capacity ratio.

Two performance measures are used to characterize vehicular LOS for a given direction of travel along an urban street segment. One measure is travel speed for through vehicles. This speed reflects the factors that influence running time along the link and the delay incurred by through vehicles at the boundary intersection. The second measure is the volume-to-capacity ratio for the through movement at the downstream boundary intersection. These performance measures indicate the degree of mobility provided by the segment. The following paragraphs characterize each service level.

LOS A describes primarily free-flow operation. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream. Control delay at the boundary intersection is minimal. The travel speed exceeds 85% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS B describes reasonably unimpeded operation. The ability to maneuver within the traffic stream is only slightly restricted, and control delay at the

boundary intersection is not significant. The travel speed is between 67% and 85% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS C describes stable operation. The ability to maneuver and change lanes at midsegment locations may be more restricted than at LOS B. Longer queues at the boundary intersection may contribute to lower travel speeds. The travel speed is between 50% and 67% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS D indicates a less stable condition in which small increases in flow may cause substantial increases in delay and decreases in travel speed. This operation may be due to adverse signal progression, high volume, or inappropriate signal timing at the boundary intersection. The travel speed is between 40% and 50% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS E is characterized by unstable operation and significant delay. Such operations may be due to some combination of adverse progression, high volume, and inappropriate signal timing at the boundary intersection. The travel speed is between 30% and 40% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS F is characterized by flow at extremely low speed. Congestion is likely occurring at the boundary intersection, as indicated by high delay and extensive queuing. The travel speed is 30% or less of the base free-flow speed, or the volume-to-capacity ratio is greater than 1.0.

Exhibit 17-2 lists the LOS thresholds established for the automobile mode on urban streets.

Exhibit 17-2
LOS Criteria: Automobile Mode

Travel Speed as a Percentage of Base Free-Flow Speed (%)	LOS by Volume-to-Capacity Ratio[a] ≤1.0	> 1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

Note: [a] Volume-to-capacity ratio of through movement at downstream boundary intersection.

Nonautomobile Modes

Historically, this manual has used a single performance measure as the basis for defining LOS. However, research documented in Chapter 5, Quality and Level-of-Service Concepts, indicates that travelers consider a wide variety of factors when they assess the quality of service provided to them. Some of these factors can be described as performance measures (e.g., speed), and others can be described as basic descriptors of the urban street character (e.g., sidewalk width). The methodology for evaluating each mode provides a procedure for mathematically combining these factors into a score. This score is then used to determine the LOS that is provided for a given direction of travel along a segment.

Exhibit 17-3 lists the scores associated with each LOS for the pedestrian travel mode. The LOS for this particular mode is determined by consideration of both the LOS score and the average pedestrian space on the sidewalk. The applicable LOS for an evaluation is determined from the table by finding the intersection of the row corresponding to the computed score value and the column corresponding to the computed space value.

Exhibit 17-3
LOS Criteria: Pedestrian Mode

Pedestrian LOS Score	LOS by Average Pedestrian Space (ft^2/p)					
	>60	>40–60	>24–40	>15–24	>8.0–15[a]	≤ 8.0[a]
≤2.00	A	B	C	D	E	F
>2.00–2.75	B	B	C	D	E	F
>2.75–3.50	C	C	C	D	E	F
>3.50–4.25	D	D	D	D	E	F
>4.25–5.00	E	E	E	E	E	F
>5.00	F	F	F	F	F	F

Note: [a] In cross-flow situations, the LOS E/F threshold is 13 ft^2/p.

The association between LOS score and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip along an urban street. The letter "A" was used to represent the "best" quality of service, and the letter "F" was used to represent the "worst" quality of service. "Best" and "worst" were left undefined, allowing the respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

Exhibit 17-4 lists the range of scores that are associated with each LOS for the bicycle and transit modes. This exhibit is also applicable for determining pedestrian LOS when a sidewalk is not available.

Exhibit 17-4
LOS Criteria: Bicycle and Transit Modes

LOS	LOS Score
A	≤2.00
B	>2.00–2.75
C	>2.75–3.50
D	>3.50–4.25
E	>4.25–5.00
F	>5.00

REQUIRED INPUT DATA

This subsection describes the required input data for the automobile, pedestrian, bicycle, and transit methodologies. Default values for some of these data are described in Section 3, Applications.

Automobile Mode

This part describes the input data needed for the automobile methodology. The data are listed in Exhibit 17-5 and are identified as "input data elements." They must be separately specified for each direction of travel on the segment and for each boundary intersection.

The last column in Exhibit 17-5 indicates whether the input data are needed for a movement group at a boundary intersection, the overall intersection, or the segment. The input data needed to evaluate the boundary intersections are identified in the appropriate chapter (i.e., Chapters 18 to 22).

The data elements listed in Exhibit 17-5 do not include variables that are considered to represent calibration factors (e.g., acceleration rate). Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Exhibit 17-5
Input Data Requirements: Automobile Mode

Data Category	Location	Input Data Element	Basis
Traffic characteristics	Boundary intersection	Demand flow rate	Movement group
	Segment	Access point flow rate	Movement group
		Midsegment flow rate	Segment
Geometric design	Boundary intersection	Number of lanes	Movement group
		Upstream intersection width	Intersection
		Turn bay length	Segment approach
	Segment	Number of through lanes	Segment
		Number of lanes at access points	Segment approach
		Turn bay length at access points	Segment approach
		Segment length	Segment
		Restrictive median length	Segment
		Proportion of segment with curb	Segment
		Number of access point approaches	Segment
Other	Segment	Analysis period duration	Segment
		Speed limit	Segment
Performance measures	Boundary intersection	Through control delay	Through-movement group
		Through stopped vehicles	Through-movement group
		2nd- and 3rd-term back-of-queue size	Through-movement group
		Capacity	Movement group
	Segment	Midsegment delay	Segment
		Midsegment stops	Segment

Notes: Movement group = one value for each turn movement with exclusive lanes and one value for the through movement (inclusive of any turn movements in a shared lane).
Through-movement group = one value for the segment through movement at the downstream boundary intersection (inclusive of any turn movements in a shared lane).
Segment = one value or condition for each direction of travel on the segment.
Segment approach = one value or condition for each intersection approach on the subject segment.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 17-5. These data describe the motorized vehicle traffic stream traveling along the street during the analysis period.

Demand Flow Rate

The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period, divided by the analysis period duration. It is expressed as an hourly flow rate, but it may represent an analysis period shorter than 1 h. Guidance for estimating this rate is provided in the chapter that corresponds to the boundary intersection configuration (i.e., Chapters 18 to 22).

Access Point Flow Rate

The access point flow rate is defined as the count of vehicles arriving at an access point intersection during the analysis period, divided by the analysis period duration. It is expressed as an hourly flow rate, but it may represent an analysis period shorter than 1 h. It should represent a demand flow rate. It is needed for all intersecting movements at each active access point intersection.

An access point approach is considered to be *active* if it has sufficient volume to have some impact on segment operations during the analysis period. As a rule of thumb, an access point approach is considered active if it has an entering flow rate of 10 vehicles per hour (veh/h) or more during the analysis period.

If the segment has many access point intersections that are considered inactive but collectively have some impact on traffic flow, those intersections can be combined into one equivalent active access point intersection. Each nonpriority movement at the equivalent access point intersection has a flow rate that is equal to the sum of the corresponding nonpriority movement flow rates of each of the individual inactive access points.

There is one exception to the aforementioned definition of access point flow rate. Specifically, if a planning analysis is being conducted in which (*a*) the projected demand coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand can be divided by the intersection peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor used should be based on local traffic peaking trends.

Midsegment Flow Rate

The midsegment flow rate is defined as the count of vehicles traveling along the segment during the analysis period, divided by the analysis period duration. It is expressed as an hourly flow rate, but it may represent an analysis period shorter than 1 h. This volume is specified separately for each direction of travel along the segment.

If one or more access point intersections exist along the segment, then the midsegment flow rate should be measured at a location between these intersections (or between an access point and boundary intersection). The location chosen should be representative in terms of its having a flow rate similar to other locations along the segment. If the flow rate is believed to vary significantly along the segment, then it should be measured at several locations and an average used in the methodology.

There is one exception to the aforementioned definition of midsegment flow rate. Specifically, if a planning analysis is being conducted in which (*a*) the projected demand coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand can be divided by the peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor used should be based on local traffic peaking trends.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 17-5. These data describe the geometric elements of the segment or intersections that are addressed in the automobile methodology.

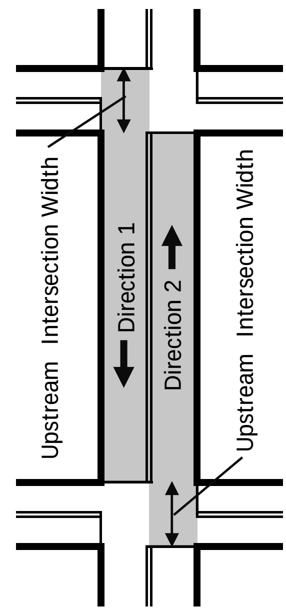
Number of Lanes

The number of lanes at the boundary intersection represents the count of lanes that are provided for each intersection traffic movement. For a turn movement, this count represents the lanes reserved for the exclusive use of turning vehicles. Turn movement lanes include turn lanes that extend backward for the length of the segment and lanes in a turn bay. Lanes that are shared by two or more movements are included in the count of through lanes and are described as *shared lanes.* If no exclusive turn lanes are provided, then the turn movement is indicated to have 0 lanes.

Upstream Intersection Width

The intersection width applies to the upstream boundary intersection for a given direction of travel and represents the effective width of the cross street. On a two-way street, it represents the distance between the stop (or yield) line for the two opposing segment through movements at the boundary intersection, as measured along the centerline of the segment. On a one-way street, it represents the distance from the stop line to the far side of the most distant traffic lane on the cross street.

Two-Way Vehicular Travel

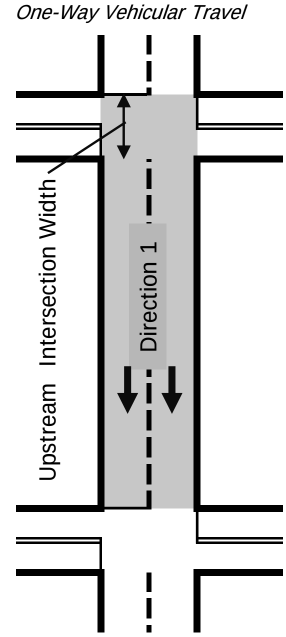


Turn Bay Length

Turn bay length represents the length of the bay at the boundary intersection for which the lanes have full width and in which queued vehicles can be stored. Bay length is measured parallel to the roadway centerline. If there are multiple lanes in the bay and they have differing lengths, then the length entered should be an average value.

If a two-way left-turn lane is provided for left-turn vehicle storage and adjacent access points exist, then the bay length entered should represent the effective storage length available to the left-turn movement. The determination of effective length is based on consideration of the adjacent access points and their associated left-turning vehicles that store in the two-way left-turn lane.

One-Way Vehicular Travel



Number of Through Lanes

The number of through lanes on the segment represents the count of lanes that extend for the length of the segment and serve through vehicles (even if a lane is dropped or added at a boundary intersection). This count is specified separately for each direction of travel along the segment. A lane provided for the exclusive use of turning vehicles is not included in this count.

Number of Lanes at Access Points

The number of lanes at an access point intersection represents the count of lanes that are provided for each traffic movement at the intersection. The method

for determining this number follows the same guidance provided in a previous paragraph for number of lanes at boundary intersections.

Turn Bay Length at Access Points

Turn bay length represents the length of the bay at the access point intersection for which the lanes have full width and in which queued vehicles can be stored. This length is needed for both segment approaches to the access point intersection. The method for determining this length follows the same guidance provided in a previous paragraph for turn bay length at boundary intersections.

Segment Length

Segment length represents the distance between the boundary intersections that define the segment. The point of measurement at each intersection is the stop line, the yield line, or the functional equivalent in the subject direction of travel. This length is measured along the centerline of the street. If it differs in the two travel directions, then an average length is used.

The *link length* is used in some calculations. It is computed as the segment length minus the width of the upstream boundary intersection.

Restrictive Median Length

The restrictive median length represents the length of street with a restrictive median (e.g., raised curb). This length is measured from median nose to median nose along the centerline of the street. It does not include the length of any median openings on the street.

Proportion of Segment with Curb

The proportion of the segment with curb represents that portion of the link length that has curb along the right side of the segment. This proportion is computed as the length of street with a curbed cross section divided by the link length. The length of street with a curbed cross section is measured from the start of the curbed cross section to the end of the curbed cross section on the link. The width of driveway openings is *not* deducted from this length. This value is input for each direction of travel along the segment.

Number of Access Point Approaches

The number of access point approaches along a segment represents the count of unsignalized driveway and public street approaches to the segment, regardless of the traffic demand entering the approach. This number is counted separately for each side of the segment. It must equal or exceed the number of active access points for which delay to segment through vehicles is computed. If the downstream boundary intersection is unsignalized, its cross-street approach on the right-hand side (in the direction of travel) is included in the count.

Other Data and Performance Measures

This subpart describes the data listed in Exhibit 17-5 that are categorized as "other data" or "performance measures."

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. Its duration is in the range of 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should use caution in interpreting the results from an analysis period of 1 h or more because the adverse impact of short peaks in traffic demand may not be detected. Also, if the analysis period is other than 15 min, then the peak hour factor should not be used.

The methodology was developed to evaluate conditions in which queue spillback does not affect the performance of the subject segment or either boundary intersection during the analysis period. If spillback affects performance, the analyst should consider using an alternative analysis tool that is able to model the effect of spillback conditions.

Operational Analysis. A 15-min analysis period should be used for operational analyses. This duration will accurately capture the adverse effects of demand peaks. Any 15-min period of interest can be evaluated with the methodology; however, a complete evaluation should always include an analysis of conditions during the 15-min period that experiences the highest traffic demand during a 24-h period.

If traffic demand exceeds capacity for a given 15-min analysis period, then a multiple-period analysis should be conducted. This type of analysis consists of an evaluation of several consecutive 15-min periods. The periods analyzed would include an initial analysis period that has no initial queue, one or more periods in which demand exceeds capacity, and a final analysis period that has no residual queue.

When a multiple-period analysis is used, segment performance measures are computed for each analysis period. Averaging performance measures across multiple analysis periods is not encouraged because it may obscure extreme values.

If a multiple-period analysis is used and the boundary intersections are signalized, then the procedure described in Chapter 18 should be used to guide the evaluation. When a procedure for multiple-period analysis is not provided in the chapter that corresponds to the boundary intersection configuration, the analyst should separately evaluate each period and use the residual queue from one period as the initial queue for the next period.

Planning Analysis. A 15-min analysis period is used for most planning analyses. However, hourly traffic demands are normally produced through the planning process. Thus, when 15-min forecast demands are not available for a 15-min analysis period, a peak hour factor must be used to estimate the 15-min demands for the analysis period. A 1-h analysis period can be used if appropriate. Regardless of analysis-period duration, a single-period analysis is typical for planning applications.

Speed Limit

Average running speed is used in the methodology to evaluate segment performance. It is correlated with speed limit when speed limit reflects the

environmental and geometric factors that have an influence on driver speed choice. As such, speed limit represents a single input variable that can be used as a convenient way to estimate running speed while limiting the need for numerous environmental and geometric input data.

The convenience of using speed limit as an input variable comes with a caution—the analyst must not infer a cause-and-effect relationship between the input speed limit and the estimated running speed. More specifically, the computed change in performance resulting from a change in the input speed limit is not likely to be indicative of performance changes that will actually be realized. Research indicates that a change in speed limit has a proportionally smaller effect on the actual average speed (*1*).

The methodology is based on the assumption that the posted speed limit is (*a*) consistent with that found on other streets in the vicinity of the subject segment and (*b*) consistent with agency policy regarding specification of speed limits. If it is known that the posted speed limit does not satisfy these assumptions, then the speed limit value that is input to the methodology should be adjusted such that it is consistent with the assumptions.

Through Control Delay

The through control delay represents the control delay to the through movement at the downstream boundary intersection. It is computed by using the appropriate procedure provided in one of Chapters 18 to 22, depending on the type of control used at the intersection.

If the intersection procedure provides delay by lane groups and the through movement is served in two or more lane groups, then the through-movement delay is computed as the weighted sum of the individual lane-group delays, where the weight for a lane group is its proportion of through vehicles.

Through Stopped Vehicles and Second- and Third-Term Back-of-Queue Size

Three variables are needed for the calculation of stop rate. These variables are needed when the downstream boundary intersection is signalized. They apply to the through-lane group at this intersection. A procedure for computing the number of fully stopped vehicles N_f, second-term back-of-queue size Q_2, and third-term back-of-queue size Q_3 is provided in Chapter 31, Signalized Intersections: Supplemental.

If the procedure provides the stop rate by lane groups and the through movement is served in two or more lane groups, then the through-movement stop rate is computed as the weighted sum of the individual lane-group stop rates, where the weight for a lane group is its proportion of through vehicles.

Capacity

The capacity of a movement group represents the maximum number of vehicles that can discharge from a queue during the analysis period, divided by the analysis period duration. This value is needed for the movements entering the segment at the upstream boundary intersection and for the movements exiting the segment at the downstream boundary intersection. With one

exception, it is computed by using the appropriate procedure provided in one of Chapters 18 to 22, depending on the type of control used at the intersection. Chapter 19, Two-Way STOP-Controlled Intersections, does not provide a procedure for estimating the capacity of the uncontrolled through movement, but this capacity can be estimated by using Equation 17-1.

Equation 17-1

$$c_{th} = 1{,}800\ (N_{th} - 1 + p^*_{0,j})$$

where

c_{th} = through-movement capacity (veh/h),

N_{th} = number of through lanes (shared or exclusive) (ln), and

$p^*_{0,j}$ = probability that there will be no queue in the inside through lane.

The probability $p^*_{0,j}$ is computed by using Equation 19-43 in Chapter 19. It is equal to 1.0 if a left-turn bay is provided for left turns from the major street.

If the procedure in Chapters 18 to 22 provides capacity by lane groups and the through movement is served in two or more lane groups, then the through-movement capacity is computed as the weighted sum of the individual lane-group capacities, where the weight for a lane group is its proportion of through vehicles. A similar approach is used to compute the capacity for a turn movement.

Midsegment Delay and Stops

Through vehicles traveling along a segment can encounter a variety of situations that cause them to slow slightly or even come to a stop. These encounters delay the through vehicles and cause their segment running time to increase. Situations that can cause this delay include

- Vehicles turning from the segment into an access point approach,
- Pedestrians crossing at a midsegment crosswalk,
- Vehicles maneuvering into or out of an on-street parking space,
- Double-parked vehicles blocking a lane, and
- Vehicles in a dropped lane that are merging into the adjacent lane.

A procedure is provided in the methodology for estimating the delay due to vehicles turning left or right into an access point approach. This edition of the HCM does not include procedures for estimating the delay or stops due to the other sources listed. If they exist on the subject segment, they must be estimated by the analyst and input to the methodology.

Nonautomobile Modes

This part describes the input data needed for the pedestrian, bicycle, and transit methodologies. The data are listed in Exhibit 17-6 and are identified as "input data elements." They must be separately specified for each direction of travel on the segment.

Exhibit 17-6
Input Data Requirements: Nonautomobile Modes

Data Category	Location	Input Data Element	Pedestrian Mode	Bicycle Mode	Transit Mode
Traffic charac-teristics	Segment, transit	Dwell time			X
		Excess wait time			X
		Passenger trip length			X
		Transit frequency			X
		Passenger load factor			X
	Segment, other	Midsegment flow rate (motorized vehicles)	X	X	
		Percent heavy vehicles		X	
		Pedestrian flow rate	X		
		Prop. of on-street parking occupied	X	X	
Geometric design	Segment, roadway	Downstream intersection width	X		
		Segment length	X	X	X
		Number of through lanes	X	X	
		Width of outside through lane	X	X	
		Width of bicycle lane	X	X	
		Width of paved outside shoulder	X	X	
		Median type and curb presence	X	X	
		No. of access point approaches		X	
	Segment, sidewalk	Presence of a sidewalk	X		
		Total walkway width	X		
		Effective width of fixed objects	X		
		Buffer width	X		
		Spacing of objects in buffer	X		
Other	Segment	Area type			X
		Pavement condition rating		X	
		Distance to nearest signal-controlled crossing	X		
		Legality of midsegment pedestrian crossing	X		
		Proportion of sidewalk adjacent to window, building, or fence	X		
	Transit stop	Transit stop location			X
		Transit stop position			X
		Proportion of stops with shelters			X
		Proportion of stops with benches			X
Performance measures	Segment	Motorized vehicle running speed	X	X	X
		Pedestrian LOS score for link			X
	Boundary intersection	Through control delay			X
		Reentry delay			X
		Effective green-to-cycle-length ratio (if signalized)			X
		Volume-to-capacity ratio (if roundabout)			X
		Pedestrian delay	X		
		Bicycle delay		X	
		Pedestrian LOS score for intersection	X		
		Bicycle LOS score for intersection		X	

Exhibit 17-6 categorizes each input data element by travel mode methodology. An “X” is used to indicate the association between a data element and methodology. A blank cell indicates that the data element is not used as input for the corresponding methodology.

The data elements listed in Exhibit 17-6 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable

values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during the presentation of the methodology.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 17-6. These data describe the vehicle, pedestrian, and transit traffic streams traveling along the segment during the analysis period. If there are multiple transit routes on the segment, then the transit-related variables are needed for each route.

Dwell Time

Dwell time represents the time that the transit vehicle is stopped at the curb to serve passenger movements, including the time required to open and close the doors. It does not include time spent stopped after passenger movements have ceased (e.g., waiting for a traffic signal or waiting for a gap in traffic to reenter the travel lane). Dwell times are typically in the range of 10 to 60 s, depending on boarding and alighting demand. Procedures for measuring and estimating dwell time are provided in the *Transit Capacity and Quality of Service Manual* (*3*).

Excess Wait Time

The scheduled departure time from a stop and the scheduled travel time for a trip set the baseline for a passenger's expectations for how long a trip should take. If the transit vehicle departs late—or worse, departs before the scheduled time (i.e., before all the passengers planning to take that vehicle have arrived at the stop)—the trip will likely take longer than planned, which negatively affects a passenger's perceptions of the quality of service.

Transit reliability is measured by *excess wait time,* the average number of minutes passengers must wait at a stop past the scheduled departure time. It is measured in the field as the sum of the differences between the scheduled and actual departure times at the preceding time point, divided by the number of transit vehicle arrivals. Early departures from the preceding time point are treated as the transit vehicle being one headway late, as a passenger arriving at the stop by the scheduled departure time would have to wait one headway for the next transit vehicle. If time point–specific excess wait time information is not available, but on-time performance (e.g., percentage of departures from a time point 0 to 5 min late) data are available for a route, then Section 2, Methodology, provides a procedure for estimating excess wait time from on-time performance.

Passenger Trip Length

The impact of a late transit vehicle departure on the overall passenger speed for a trip (as measured by using scheduled departure time to actual arrival time) depends on the length of the passenger's trip. For example, a departure 5 min late has more of a speed impact on a 1-mi-long trip than on a 10-mi-long trip. Average passenger trip length is used to determine the impact of late departures on overall trip speed. For most purposes, the average trip length can be determined from National Transit Database data for the transit agency (*4*) by dividing total passenger-miles by total unlinked trips. However, if an analyst has

reason to believe that average trip length on a route is substantially different from the system average, a route-specific value can be determined from automatic passenger counter data or National Transit Database count sheets for the route by dividing total passenger-miles by the total number of boarding passengers.

Transit Frequency

Transit frequency is defined as the count of scheduled fixed-route transit vehicles that stop on or near the segment during the analysis period. It is expressed in units of transit vehicles per hour.

Scheduled transit vehicles can be considered "local" or "nonlocal." Local transit vehicles make regular stops along the street (typically every 0.25 mi or less), although they do not necessarily stop within the analysis segment when segment lengths are short or when stops alternate between the near and far sides of boundary intersections. They are always counted, regardless of whether they stop within the subject segment. Nonlocal transit vehicles operate on routes with longer stop spacing than local routes (e.g., limited-stop, bus rapid transit, or express routes). They are only counted when they stop within the subject segment.

Passenger Load Factor

The load factor represents the number of passengers occupying the transit vehicle divided by the number of seats on the vehicle. If the number of passengers equals the number of seats, then the load factor equals 1.0. This factor should be measured in the field or obtained from the agency serving the transit route. It is an average value for all of the scheduled fixed-route transit vehicles that travel along the segment during the analysis period.

Midsegment Flow Rate

The midsegment flow rate of motorized vehicles is equivalent to the midsegment flow rate defined previously for the automobile mode.

Percent Heavy Vehicles

A heavy vehicle is defined as any vehicle with more than four tires touching the pavement. Local buses that stop within the intersection area are not included in the count of heavy vehicles. The percentage of heavy vehicles represents the count of heavy vehicles that arrive during the analysis period divided by the total vehicle count for the same period. This percentage is provided for the same location on the segment as represented by the midsegment flow rate.

Pedestrian Flow Rate

The pedestrian flow rate is based on the count of pedestrians traveling along the outside of the subject segment during the analysis period. A separate count is taken for each direction of travel along the side of the segment. Each count is divided by the analysis period duration to yield a directional hourly flow rate. These rates are then added to obtain the pedestrian flow rate for that side.

Proportion of On-Street Parking Occupied

This variable represents the proportion of the segment's right-hand curb line on which parked vehicles are present during the analysis period. It is computed as the sum of the curb-line lengths occupied by parked vehicles divided by the link length. Also, the use of pavement markings to delineate the parking lane should be noted.

If parking is not allowed on the segment, then the proportion equals 0.0. If parking is allowed along the segment but the spaces are not used during the analysis period, then the proportion equals 0.0. If parking is allowed along the full length of the segment but only one-half of the spaces are occupied during the analysis period, then the proportion equals 0.50.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 17-6. These data describe the geometric elements that influence pedestrian, bicycle, or transit performance. All input data should be representative of the segment for its entire length. An average value should be used for each element that varies along the segment. Segment length, number of through lanes, and number of access point approaches are defined previously for the automobile mode.

Downstream Intersection Width

The intersection width applies to the downstream boundary intersection for a given direction of travel and represents the effective width of the cross street. On a two-way street, it represents the distance between the stop (or yield) line for the two opposing segment through movements at the boundary intersection, as measured along the centerline of the segment. On a one-way street, it represents the distance from the stop line to the far side of the most distant traffic lane on the cross street.

Width of Outside Through Lane, Bicycle Lane, and Paved Outside Shoulder

The widths of several individual elements of the cross section are considered input data. These elements include the outside lane that serves motorized vehicles traveling along the segment, the bicycle lane adjacent to the outside lane (if used), and the outside shoulder. The outside shoulder may be used for on-street parking. The width of each of these elements is mutually exclusive because they are adjacent (i.e., not overlapped) in the cross section.

The outside lane width does not include the width of the gutter. If curb and gutter are present, then the width of the gutter is included in the shoulder width (i.e., shoulder width is measured to the curb face when a curb is present).

Median Type and Curb Presence

The median type is designated as undivided, nonrestrictive (e.g., two-way left-turn lane), or restrictive (e.g., raised curb). Whether the cross section has curb on the outside edge of the roadway should also be noted.

Presence of a Sidewalk

A sidewalk is a paved walkway that is provided at the side of the roadway. It is assumed that pedestrians will walk in the street if a sidewalk is not present.

Total Walkway Width

Total walkway width is measured from the outside edge of the road pavement (or face of curb, if present) to the far edge of the sidewalk (as sometimes delineated by a building face or landscaping). It includes the width of any buffer (see below), if present. If this width varies along the segment, then an average value is used. A paved shoulder is not included in this width measurement.

Effective Width of Fixed Objects

Two input variables are used to describe fixed objects along the walkway. One variable represents the effective width of objects along the inside of the sidewalk. These objects include light poles, traffic signs, planter boxes, and so forth. Typical widths for these objects are provided in Chapter 23, Off-Street Pedestrian and Bicycle Facilities. All objects along the sidewalk should be considered and an average value for the length of the sidewalk input to the methodology.

The second variable represents the effective width of objects along the outside of the sidewalk. It is determined in the same manner as was the first variable.

Buffer Width and Spacing of Objects in Buffer

The buffer width represents the distance between the outside edge of the paved roadway (or face of curb, if present) and the near edge of the sidewalk. This element of the cross section is not designed for use by pedestrians or motorized vehicles. It may be unpaved or include various vertical objects that are continuous (e.g., barrier) or discontinuous (e.g., trees, bollards) to prevent pedestrian use. If vertical objects are in the buffer, then the average spacing of those objects that are 3 ft or more in height should also be recorded.

Other Data

This subpart describes the data listed in Exhibit 17-6 that are categorized as "other data."

Area Type

Area type describes the environment in which the subject segment is located. This data element is used in the transit methodology to set a baseline for passenger expectations of typical transit travel speeds. For this application, it is sufficient to indicate whether the area type is a "central business district of a metropolitan area with over 5 million persons" or "other."

Pavement Condition Rating

The pavement condition rating describes the road surface in terms of ride quality and surface defects. It is based on the Present Serviceability Rating, a

subjective rating system based on a scale of 0 to 5 (*5*). Exhibit 17-7 provides a description of pavement conditions associated with various ratings.

Distance to Nearest Signal-Controlled Crossing

This input variable is needed if there is an identifiable pedestrian path (*a*) that intersects the segment and continues on beyond the segment and (*b*) on which most crossing pedestrians travel. This variable defines the distance pedestrians must travel along the segment should they divert from the path to cross the segment at the nearest signalized crossing. The crossing will typically be at a signalized intersection. However, it may also be at a signalized crosswalk provided at a midsegment location. If the crossing is at a signalized intersection, it will likely occur in the crosswalk on the side of the intersection that is nearest to the segment. Occasionally, it will be on the far side of the intersection because the near-side crosswalk is closed (or a crossing at this location is otherwise prohibited). This distance is measured along one side of the subject segment; the methodology accounts for the return distance once the pedestrian arrives at the other side of the segment.

Exhibit 17-7
Pavement Condition Rating

Pavement Condition Rating	Pavement Description	Motorized Vehicle Ride Quality and Traffic Speed
4.0 to 5.0	New or nearly new superior pavement. Free of cracks and patches.	Good ride.
3.0 to 4.0	Flexible pavements may begin to show evidence of rutting and fine cracks. Rigid pavements may begin to show evidence of minor cracking.	Good ride.
2.0 to 3.0	Flexible pavements may show rutting and extensive patching. Rigid pavements may have a few joint fractures, faulting, or cracking.	Acceptable ride for low-speed traffic but barely tolerable for high-speed traffic.
1.0 to 2.0	Distress occurs over 50% or more of the surface. Flexible pavement may have large potholes and deep cracks. Rigid pavement distress includes joint spalling, patching, and cracking.	Pavement deterioration affects the speed of free-flow traffic. Ride quality not acceptable.
0.0 to 1.0	Distress occurs over 75% or more of the surface. Large potholes and deep cracks exist.	Passable only at reduced speed and considerable rider discomfort.

Legality of Midsegment Pedestrian Crossing

This input indicates whether a pedestrian can cross the segment at any point along its length, regardless of location. If it is illegal to make this crossing at any point, then the pedestrian is assumed to be required to divert to the nearest signalized intersection to cross the segment.

Proportion of Sidewalk Adjacent to Window, Building, or Fence

Three proportions are input for a sidewalk. One proportion represents the length of sidewalk adjacent to a fence or low wall divided by the length of the link. The second proportion represents the length of the sidewalk adjacent to a building face divided by the length of the link. The final proportion represents

the length of the sidewalk adjacent to a window display divided by the length of the link.

Transit Stop Location

This input describes whether a transit stop is located on the near side of a boundary intersection or elsewhere. A portion of the time required to serve a near-side transit stop at a boundary intersection may overlap with the control delay incurred at the intersection.

Transit Stop Position

Transit stops can be either *on-line*, where the bus stops entirely or mostly in the travel lane and does not have to yield to other vehicles upon exiting the stop, or *off-line,* where the bus pulls out of the travel lane to serve the stop and may have to yield to other vehicles upon exiting.

Proportion of Stops with Shelters and with Benches

These two input data describe the passenger amenities provided at a transit stop. A sheltered stop provides a structure with a roof and three enclosed sides that protect occupants from wind, rain, and sun. A shelter with a bench is counted twice, once as a shelter and a second time as a bench.

Performance Measures

This subpart describes the data listed in Exhibit 17-6 that are categorized as "performance measures." The through control delay variable was previously described for the automobile mode (in Exhibit 17-5).

Motorized Vehicle Running Speed

The motorized vehicle running speed is used in all of the nonautomobile methodologies. It is based on the segment running time obtained from the automobile methodology. The running speed is equal to the segment length divided by the segment running time.

Pedestrian LOS Score for Link

The pedestrian LOS score for the link is used in the transit methodology. It is obtained from the pedestrian methodology in this chapter.

Reentry Delay

The final component of transit vehicle stop delay is the reentry delay, the time (in seconds) a transit vehicle spends waiting for a gap to reenter the adjacent traffic stream. Reentry delay is estimated as follows (*3*):

- Reentry delay is zero at on-line stops.
- At off-line stops away from the influence of a signalized intersection queue, reentry delay is estimated from the procedures of Chapter 19, Two-Way STOP-Controlled Intersections, as if the bus were making a right turn onto the link, but a critical headway of 7 s is used to account for the slower acceleration of buses.

- At an off-line bus stop located within the influence of a signalized intersection queue, reentry delay is estimated from the queue service time, g_s, by using the procedures of Chapter 18, Signalized Intersections.

Reentry delay can be reduced by the presence of yield-to-bus laws or placards (and motorist compliance with them), the existence of an acceleration lane or queue jump departing a stop, or a higher-than-normal degree of bus driver aggressiveness in forcing buses back into the traffic stream. Analyst judgment and local data can be used to make appropriate adjustments to reentry delay in these cases.

Effective Green-to-Cycle-Length Ratio

The effective green-to-cycle-length ratio for the through movement is used in the transit methodology when the boundary intersection is a traffic signal and has a near-side transit stop. It is obtained from the Chapter 18 methodology.

Volume-to-Capacity Ratio (If Roundabout)

If the boundary intersection is a roundabout and it has a near-side transit stop, then the volume-to-capacity ratio for the rightmost lane of the segment approach to the roundabout is needed. It is obtained from the Chapter 21 methodology.

Pedestrian Delay

Three pedestrian delay variables are needed. The first is the delay to pedestrians who travel through the boundary intersection along a path that is parallel to the segment centerline. The pedestrian movement of interest is traveling on the subject side of the street and heading in a direction that is "with" or "against" the motorized traffic stream. For a two-way STOP-controlled boundary intersection, this delay is reasoned to be negligible. For a signal-controlled boundary intersection, the procedure described in Chapter 18 is used to compute this delay.

The second delay variable needed describes the delay incurred by pedestrians who cross the subject segment at the *nearest* signal-controlled crossing. If the nearest crossing is at a signalized intersection, then the procedure described in Chapter 18 is used to compute this delay. If the nearest crossing is at a midsegment signalized crosswalk, then this delay should equal the pedestrian's average wait for service after pressing the pedestrian push button. This wait will depend on the signal settings and could range from 5 to 25 s/pedestrian (s/p).

The third delay variable needed is the pedestrian waiting delay. This delay is incurred when pedestrians wait at an uncontrolled crossing location. If this type of crossing is legal, then the pedestrian waiting delay is determined by using the procedure in Chapter 19, Two-Way STOP-Controlled Intersections. If it is illegal, then the pedestrian waiting delay does not need to be calculated.

Bicycle Delay

Bicycle delay is the delay to bicyclists who travel through the boundary intersection along a path that is parallel to the segment centerline. The bicycle

movement of interest is traveling on the subject side of the street and heading in the same direction as motorized vehicles. For a two-way STOP-controlled boundary intersection, this delay is reasoned to be negligible. For a signal-controlled boundary intersection, the procedure described in Chapter 18 is used to compute this delay.

Pedestrian LOS Score for Intersection

The pedestrian LOS score for the signalized intersection is used in the pedestrian methodology. It is obtained from the pedestrian methodology in Chapter 18.

Bicycle LOS Score for Intersection

The bicycle LOS score for the signalized intersection is used in the bicycle methodology. It is obtained from the bicycle methodology in Chapter 18.

SCOPE OF THE METHODOLOGY

Four methodologies are presented in this chapter. One methodology is provided for each of the automobile, pedestrian, bicycle, and transit modes. This section identifies the conditions for which each methodology is applicable.

- **Signalized and two-way STOP-controlled boundary intersections.** All methodologies can be used to evaluate segment performance with signalized or two-way STOP-controlled boundary intersections. In the latter case, the cross street is STOP controlled. The automobile methodology can also be used to evaluate performance with all-way STOP- or YIELD-controlled (e.g., roundabout) boundary intersections.
- **Arterial and collector streets.** The four methodologies were developed with a focus on arterial and collector street conditions. If a methodology is used to evaluate a local street, then the performance estimates should be carefully reviewed for accuracy.
- **Steady flow conditions.** The four methodologies are based on the analysis of steady traffic conditions and, as such, are not well suited to the evaluation of unsteady conditions (e.g., congestion, queue spillback, signal preemption).
- **Target road users.** Collectively, the four methodologies were developed to estimate the LOS perceived by automobile drivers, pedestrians, bicyclists, and transit passengers. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers, automobile passengers, delivery truck drivers, or recreational vehicle drivers). However, it is likely that the perceptions of these other road users are reasonably well represented by the road users for whom the methodologies were developed.
- **Target travel modes.** The automobile methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The pedestrian, bicycle, and transit methodologies address travel by walking, bicycle, and transit vehicle, respectively. The transit methodology is

limited to the evaluation of public transit vehicles operating in mixed or exclusive traffic lanes and stopping along the street. The methodologies are not designed to evaluate the performance of other travel means (e.g., grade-separated rail transit, golf carts, or motorized bicycles).

- **Influences in the right-of-way.** A road user's perception of quality of service is influenced by many factors inside and outside of the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside of the right-of-way (e.g., buildings, parking lots, scenery, or landscaped yards) that might influence a traveler's perspective. This approach was followed because factors outside of the right-of-way are not under the direct control of the agency operating the street.
- **Mobility focus for automobile methodology.** The automobile methodology is intended to facilitate the evaluation of mobility. Accessibility to adjacent properties by way of automobile is not directly evaluated with this methodology. Regardless, a segment's accessibility should also be considered in evaluating its performance, especially if the segment is intended to provide such access. Oftentimes, factors that favor mobility reflect minimal levels of access and vice versa.
- **"Typical pedestrian" focus for pedestrian methodology.** The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. As such, the performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to the Americans with Disabilities Act requirements. For this reason, they should not be considered as a substitute for a formal compliance assessment of a pedestrian facility.

LIMITATIONS OF THE METHODOLOGY

In general, the methodologies described in this chapter can be used to evaluate the performance of most traffic streams traveling along an urban street segment. However, the methodologies do not address all traffic conditions or types of control. The inability to replicate the influence of a condition or control type in the methodology represents a limitation. This subsection identifies the known limitations of the methodologies described in this chapter. If one or more of these limitations is believed to have an important influence on the performance of a specific street segment, then the analyst should consider using alternative methods or tools.

Automobile Modes

The automobile methodology does not directly account for the effect of the following conditions on street segment operation:

- On-street parking activity along the link (note that on-street parking activity on the approach to a signalized boundary intersection is addressed in Chapter 18, Signalized Intersections),
- Significant grade along the link,

- Capacity constraints between intersections (e.g., narrow bridges),
- Queuing at the downstream boundary intersection consistently backing up to and interfering with the operation of the upstream intersection or an access point intersection,
- Stops incurred by segment through vehicles as a result of a vehicle ahead turning from the segment into an access point,
- Bicycles sharing a traffic lane with vehicular traffic, and
- Cross-street congestion or a railroad crossing that blocks through traffic.

In addition, any limitations associated with the methodologies used to evaluate the intersections that bound the urban street segment are shared with this methodology. These limitations are listed in Chapters 18 to 22.

Nonautomobile Modes

This part identifies the limitations of the pedestrian, bicycle, and transit methodologies. These methodologies are not able to model the presence of railroad crossings. In addition, the pedestrian methodology does not model the following conditions:

- Segments bounded by all-way STOP-controlled intersections or roundabouts;
- Midsegment unsignalized crosswalks;
- Grades in excess of 2%;
- Pedestrian overcrossings for service across or along the segment;
- Points of high-volume pedestrian access to a sidewalk, such as a transit stop or a doorway from a large office building; and
- Points where a high volume of vehicles cross the sidewalk, such as a parking garage entrance.

In addition, the bicycle methodology is not able to model the following conditions:

- Segments bounded by all-way STOP-controlled intersections or roundabouts, and
- Grades in excess of 2%.

With regard to the first bullet point in each of the two lists above, procedures have not been developed yet to address the effect of all-way STOP control or YIELD control on intersection performance from a pedestrian or bicyclist perspective.

2. METHODOLOGY

OVERVIEW

This section describes four methodologies for evaluating the performance of an urban street segment. Each methodology addresses one possible travel mode within the street right-of-way. Analysts should choose the combination of methodologies that are appropriate for their analysis needs.

A complete evaluation of segment operation includes the separate examination of performance for all relevant travel modes for each travel direction. The performance measures associated with each mode and travel direction are assessed independently of one another. They are not mathematically combined into a single indicator of segment performance. This approach ensures that all performance impacts are considered on a mode-by-mode and direction-by-direction basis.

The focus of each methodology in this chapter is the segment. Methodologies for quantifying the performance of the downstream boundary intersection are described in other chapters (i.e., Chapters 18 to 22). The methodology described in Chapter 16, Urban Street Facilities, can be used to combine the performance measures (for a specified travel mode) on successive segments into an overall measure of facility performance for each mode and travel direction.

AUTOMOBILE MODE

This subsection provides an overview of the methodology for evaluating urban street segment performance from the motorist's perspective. The methodology is computationally intense and requires software to implement. The intensity stems from the need to model the traffic movements that enter or exit the segment in terms of their interaction with each other and with the traffic control elements of the boundary intersection. Default values are provided in Section 3, Applications, to support planning analyses for which the required input data are not available.

A Quick Estimation Method for evaluating segment performance at a planning level of analysis is provided in Chapter 30, Urban Street Segments: Supplemental. This method is not computationally intense and can be applied by using hand calculations.

The methodology is used to evaluate automobile performance on an urban street segment. Each travel direction along the segment is separately evaluated. *Unless otherwise stated, all variables are specific to the subject direction of travel.*

The methodology has been developed to evaluate automobile performance for a street segment bounded by intersections that can have a variety of control types. The focus of the discussion in this subsection is on the use of the methodology to evaluate a coordinated signal system because this type of control is the most complex. However, as appropriate, the discussion is extended to describe how key elements of this methodology can be used to evaluate automobile performance in noncoordinated systems.

Because of the intensity of the computations for coordinated–actuated control, the objective of this subsection is to introduce the analyst to the calculation process and to discuss the key analytic procedures. This objective is achieved by outlining the procedures that make up the methodology while highlighting important equations, concepts, and interpretations. A more detailed discussion of these procedures is provided in Chapter 30, Urban Street Segments: Supplemental.

The computational engine developed by the Transportation Research Board Committee on Highway Capacity and Quality of Service represents the most detailed description of this methodology. Additional information about this engine is provided in Chapter 30.

Framework

Exhibit 17-8 illustrates the calculation framework of the automobile methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. These calculations are described more fully in the remainder of this subsection.

The framework illustrates the calculation process as applied to two system types: coordinated and noncoordinated. The analysis of coordinated systems recognizes the influence of an upstream signalized intersection on the performance of the street segment. The analysis of noncoordinated systems is based on the assumption that arrivals to a boundary intersection are random.

The framework is further subdivided into the type of traffic control used at the intersections that bound the segment. This approach recognizes that a boundary intersection can be signalized, two-way STOP-controlled, all-way STOP-controlled, or a roundabout. Although not indicated in the exhibit, the boundary intersection could also be an interchange ramp terminal.

There is reference in Exhibit 17-8 to various procedures described in Chapters 18, 20, and 21. With regard to Chapter 18, the procedure for computing actuated phase duration is needed for the analysis of actuated intersections on both coordinated and noncoordinated segments. Also, the procedure for computing control delay in Chapter 18 is needed for the estimation of segment through-movement delay. The delay estimation procedure for roundabouts and all-way STOP-controlled intersections is needed from their respective chapters for the analysis of noncoordinated segments.

Performance measures estimated for each segment travel direction include

- Travel speed,
- Stop rate, and
- Automobile traveler perception score.

The perception score is derived from traveler perception research and is an indication of travelers' relative satisfaction with service provided along the segment.

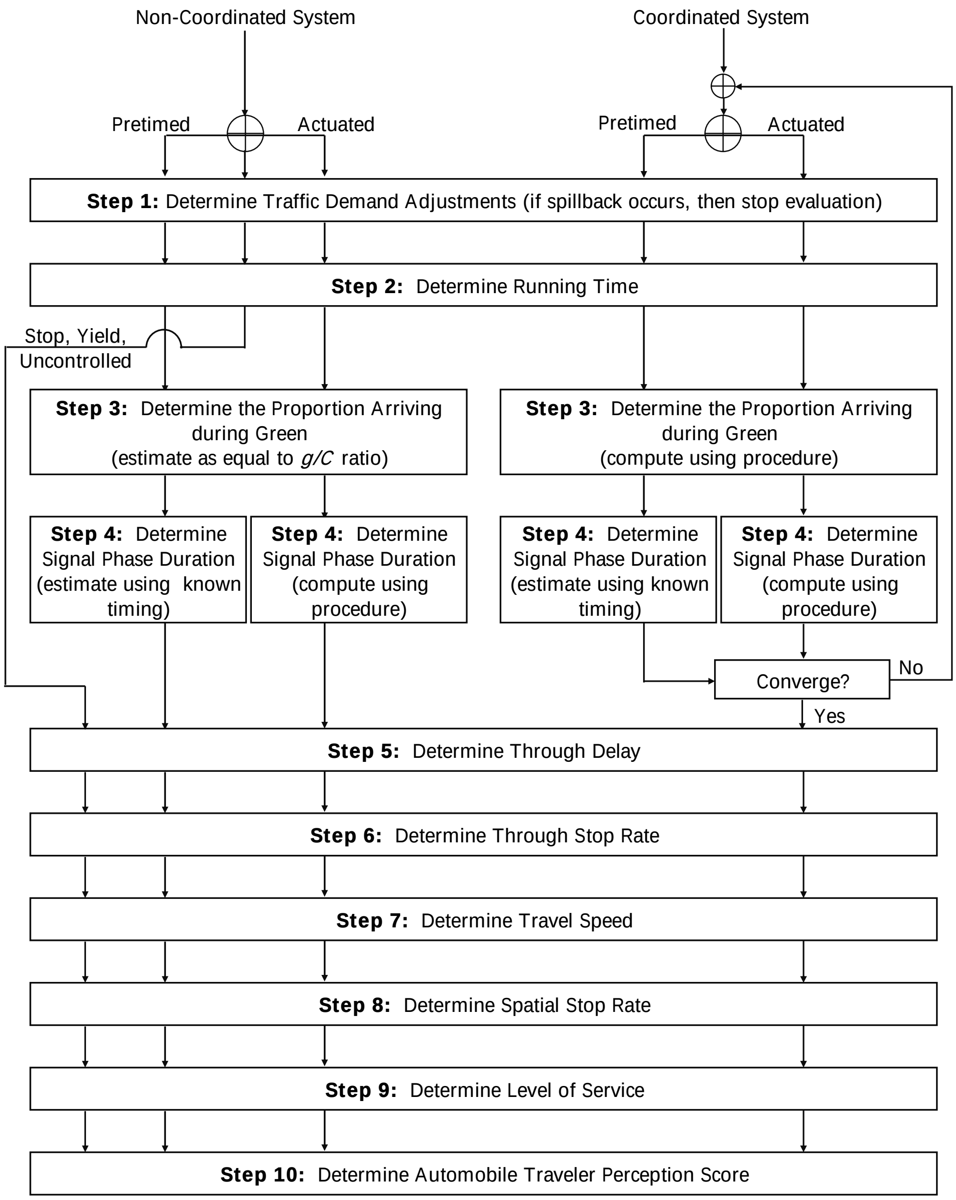


Exhibit 17-8
Automobile Methodology for Urban Street Segments

Step 1: Determine Traffic Demand Adjustments

During this step, various adjustments are undertaken to ensure the volumes evaluated accurately reflect segment traffic conditions. The adjustments include (*a*) limiting entry to the segment due to capacity constraint, (*b*) balancing the volumes entering and exiting the segment, and (*c*) mapping entry-to-exit flow paths by using an origin–destination matrix. Also during this step, a check is made for the occurrence of spillback from a turn bay or from one segment into another segment. As indicated in Exhibit 17-8, the evaluation should not proceed if spillback occurs because the methodology does not address this condition.

The procedures for making these adjustments and checks are described in Chapter 30. These adjustments and checks are not typically used for planning and preliminary engineering analyses.

Capacity Constraint

When the demand volume for an intersection traffic movement exceeds its capacity, the discharge volume from the intersection is restricted (or metered). When this metering occurs for a movement that enters the subject segment, the volume arriving at the downstream signal is reduced below the unrestricted value.

To determine whether metering occurs, the capacity of each upstream movement that discharges into the subject segment must be computed and then checked against the associated demand volume. If this volume exceeds movement capacity, then the volume entering the segment must be reduced to equal the movement capacity.

Volume Balance

Volume balance describes a condition in which the combined volume from all movements entering a segment equals the combined volume exiting the segment, in a given direction of travel. The segment is balanced when entering volume equals exit volume for both directions of travel. Unbalanced volumes often exist in turn movement counts when the count at one intersection is taken at a different time than the count at the adjacent intersection. They are also likely to exist when access point intersections exist but their volume is not counted.

The accuracy of the performance evaluation may be adversely affected if the volumes are not balanced. The extent of the impact is based on the degree to which the volumes are unequal. To balance the volumes, the methodology assumes that the volume for each movement entering the segment is correct and adjusts the volume for each movement exiting the segment in a proportional manner such that a balance is achieved. The exiting volumes computed in this manner represent a best estimate of the actual *demand* volumes, such that the adjustment process does not preclude the possibility of queue buildup by one or more exit movements at the downstream boundary intersection during the analysis period.

Origin–Destination Distribution

The volume of traffic that arrives at a downstream intersection for a given downstream movement represents the combined volume from each upstream point of entry weighted by its percentage contribution to the downstream movement. The distribution of these contribution percentages between each upstream and downstream pair is represented as an origin–destination distribution matrix.

The concept of an origin–destination distribution matrix is illustrated by example. Consider the segment shown in Exhibit 17-9. There are three entry volumes at upstream Intersection A that contribute to three exit volumes at downstream Intersection B. There is also an entrance and exit volume at the access point intersection located between the two intersections. It should be noted that 1,350 veh/h enter the segment and 1,350 veh/h exit the segment; thus there is volume balance for this example segment. The origin–destination distribution matrix for this sample street segment is shown in Exhibit 17-10.

Exhibit 17-9
Entry and Exit Volume on Example Segment

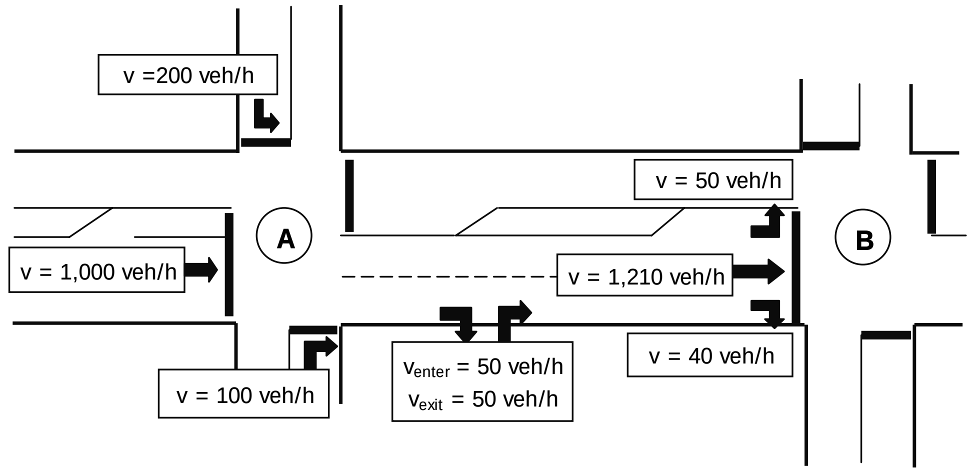


Exhibit 17-10
Example Origin–Destination Distribution Matrix

Origin Volume by Movement (veh/h)				Destination Volume	
Left	Through	Right	Access Point	Movement	Total Volume (veh/h)
2	46	2	0	Left	50
188	877	95	50	Through	1,210
3	36	1	0	Right	40
7	41	2	0	Access Point	50
200	1,000	100	50		1,350

The column totals in the last row of Exhibit 17-10 correspond to the entry volumes shown in Exhibit 17-9. The row totals in the last column of Exhibit 17-10 indicate the exit volumes. The individual cell values indicate the volume contribution of each upstream movement to each downstream movement. For example, of the 1,000 through vehicles that enter the segment, 877 depart the segment as a through movement, 46 depart as a left-turn movement, and so on. The volumes in the individual cells are sometimes expressed as a proportion of the column total.

The automobile methodology computes one origin–destination matrix for movements between the upstream boundary intersection and a downstream junction (i.e., either an access point or the downstream boundary intersection). When the boundary intersections are signalized, the matrix for movements between the upstream and downstream boundary intersections is used to compute the proportion of vehicles arriving during the green indication for each exit movement. The matrix for movements between the upstream boundary intersection and a downstream access point is used to compute the proportion of time that a platoon is passing through the access point and effectively blocking nonpriority movements from entering or crossing the street.

Spillback Occurrence

Segment spillback can be characterized as one of two types: cyclic and sustained. Cyclic spillback occurs when the downstream boundary intersection is signalized and its queue backs into the upstream intersection as a result of queue growth during the red indication. When the green indication is presented, the queue dissipates and spillback is no longer present for the remainder of the cycle.

This type of spillback can occur on short street segments with relatively long signal cycle lengths.

Sustained spillback occurs at some point during the analysis period and is a result of oversaturation (i.e., more vehicles discharging from the upstream intersection than can be served at the subject downstream intersection). The queue does not dissipate at the end of each cycle. Rather, it remains present until the downstream capacity is increased or the upstream demand is reduced.

The preceding discussion has focused on segment spillback; however, the concepts are equally applicable to turn bay spillback. In this case, the queue of turning vehicles exceeds the bay storage and spills back into the adjacent lane that is used by other vehicular movements. The occurrence of both segment and bay spillback must be checked during this step.

Use of this methodology to evaluate segments (or intersection turn bays) with significant, sustained spillback is problematic because of the associated unsteady conditions and complex interactions. The procedure described in Chapter 30 is used in this step to compute the time when sustained spillback occurs, if it occurs. If this time of occurrence is shorter than the analysis period, then the methodology may not yield accurate performance estimates. In this situation, the analyst should consider either (*a*) reducing the analysis period such that it ends before spillback occurs or (*b*) using an alternative analysis tool that is able to model the effect of spillback conditions.

Step 2: Determine Running Time

A procedure for determining segment running time is described in this step. This procedure includes the calculation of free-flow speed, a vehicle proximity adjustment factor, and the additional running time due to midsegment delay sources. Each calculation is discussed in the following subparts, which culminate with the calculation of segment running time.

A. Determine Free-Flow Speed

Free-flow speed represents the average running speed of through automobiles traveling along a segment under low-volume conditions and not delayed by traffic control devices or other vehicles. It reflects the effect of the street environment on driver speed choice. Elements of the street environment that influence this choice under free-flow conditions include speed limit, access point density, median type, curb presence, and segment length.

The determination of free-flow speed is based on the calculation of base free-flow speed and an adjustment factor for signal spacing. These calculations are described in the next few paragraphs, which culminate in the calculation of free-flow speed.

Base Free-Flow Speed

The base free-flow speed is defined to be the free-flow speed on longer segments. It includes the influence of speed limit, access point density, median type, and curb presence. It is computed by using Equation 17-2. Alternatively, it can be measured in the field by using the technique described in Chapter 30.

$$S_{fo} = S_0 + f_{CS} + f_A$$

Equation 17-2

where

S_{fo} = base free-flow speed (mi/h),

S_0 = speed constant (mi/h),

f_{CS} = adjustment for cross section (mi/h), and

f_A = adjustment for access points (mi/h).

The speed constant and adjustment factors used in Equation 17-2 are listed in Exhibit 17-11. Equations provided in the table footnote can also be used to compute these adjustment factors.

Exhibit 17-11
Base Free-Flow Speed Adjustment Factors

Speed Limit (mi/h)	Speed Constant S_0 (mi/h)[a]	Median Type	Percent with Restrictive Median (%)	Adjustment for Cross Section f_{CS} (mi/h)[b] No Curb	Curb
25	37.4	Restrictive	20	0.3	-0.9
30	39.7		40	0.6	-1.4
35	42.1		60	0.9	-1.8
40	44.4		80	1.2	-2.2
45	46.8		100	1.5	-2.7
50	49.1	Nonrestrictive	Not applicable	0.0	-0.5
55	51.5	No median	Not applicable	0.0	-0.5

Access Density D_a (points/mi)	Adjustment for Access Points f_A by Lanes N_{th} (mi/h)[c] 1 Lane	2 Lanes	3 Lanes	4 Lanes
0	0.0	0.0	0.0	0.0
2	-0.2	-0.1	-0.1	0.0
4	-0.3	-0.2	-0.1	-0.1
10	-0.8	-0.4	-0.3	-0.2
20	-1.6	-0.8	-0.5	-0.4
40	-3.1	-1.6	-1.0	-0.8
60	-4.7	-2.3	-1.6	-1.2

Notes: [a] $S_0 = 25.6 + 0.47 S_{pl}$ where S_{pl} = posted speed limit (mi/h).
[b] $f_{CS} = 1.5\, p_{rm} - 0.47\, p_{curb} - 3.7\, p_{curb}\, p_{rm}$ where p_{rm} = proportion of link length with restrictive median (decimal) and p_{curb} = proportion of segment with curb on the right-hand side (decimal).
[c] $f_A = -0.078\, D_a / N_{th}$ with $D_a = 5{,}280\,(N_{ap,s} + N_{ap,o})/(L - W_i)$ where D_a = access point density on segment (points/mi); N_{th} = number of through lanes on the segment in the subject direction of travel (ln); $N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points); $N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points); and W_i = width of signalized intersection (ft).

Adjustment for Signal Spacing

Empirical evidence suggests that a shorter segment length (when defined by signalized boundary intersections) tends to influence the driver's choice of free-flow speed (*1*). Shorter segments have been found to have a slower free-flow speed, all other factors being the same. Equation 17-3 is used to compute the value of an adjustment factor that accounts for this influence.

$$f_L = 1.02 - 4.7 \frac{S_{fo} - 19.5}{\max(L_s, 400)} \quad \leq 1.0$$

Equation 17-3

where

f_L = signal spacing adjustment factor,

S_{fo} = base free-flow speed (mi/h), and

L_s = distance between adjacent signalized intersections (ft).

Equation 17-3 was derived by using signalized boundary intersections. For more general applications, the definition of distance L_s is broadened such that it equals the distance between the two intersections that (*a*) bracket the subject segment and (*b*) each have a type of control that can impose on the subject through movement a legal requirement to stop or yield.

Free-Flow Speed

Free-flow speed is computed by using Equation 17-4 on the basis of estimates of base free-flow speed and the signal spacing adjustment factor. Alternatively, it can be entered directly by the analyst. It can also be measured in the field by using the technique described in Chapter 30.

Equation 17-4

$$S_f = S_{fo} f_L$$

where S_f equals the free-flow speed (mi/h) and other variables are as previously defined.

B. Compute Adjustment for Vehicle Proximity

The proximity adjustment factor adjusts the free-flow running time to account for the effect of traffic density. The adjustment results in an increase in running time (and corresponding reduction in speed) with an increase in volume. The reduction in speed is a result of shorter headways associated with the higher volume and drivers' propensity to be more cautious when headways are short. Equation 17-5 is used to compute the proximity adjustment factor.

Equation 17-5

$$f_v = \frac{2}{1 + \left(1 - \frac{v_m}{52.8\, N_{th}\, S_f}\right)^{0.21}}$$

where

f_v = proximity adjustment factor,

v_m = midsegment demand flow rate (veh/h),

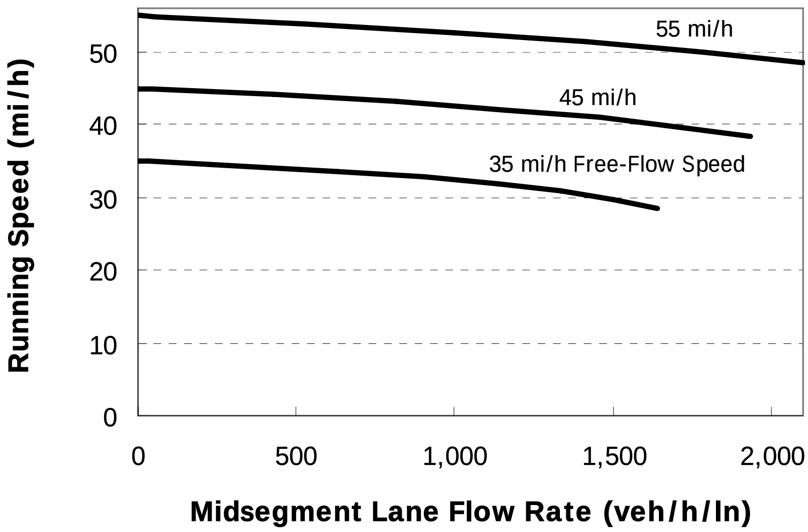
N_{th} = number of through lanes on the segment in the subject direction of travel (ln), and

S_f = free-flow speed (mi/h).

The relationship between running speed [= (3,600 L)/(5,280 t_R), where L is the segment length in feet and t_R is the segment running time in seconds] and volume for an urban street segment is shown in Exhibit 17-12. Trend lines are shown for three specific free-flow speeds. At a flow rate of 1,000 vehicles per hour per lane (veh/h/ln), each trend line shows a reduction of about 2.5 mi/h relative to the free-flow speed. The trend lines extend beyond 1,000 veh/h/ln. However, it is unlikely that a volume in excess of this amount will be experienced on a segment bounded by intersections at which the through movement is regulated by a traffic control device.

Exhibit 17-12
Speed–Flow Relationship for Urban Street Segments

LIVE GRAPH
Click here to view



C. Compute Delay due to Turning Vehicles

Vehicles turning from the subject street segment into an access point approach can cause a delay to following through vehicles. For right-turn vehicles, the delay results when the following vehicles' speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point intersection. Delay due to left-turning vehicles occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane. A procedure for computing this delay at each access point intersection is described in Chapter 30.

For planning and preliminary engineering analyses, Exhibit 17-13 can be used to estimate the delay due to turning vehicles at one representative access point intersection by using a midsegment volume that is typical for all such access points. The values in the exhibit represent the delay to through vehicles due to left and right turns at one access point intersection. The selected value is multiplied by the number of access point intersections on the segment to estimate delay due to left and right turns (= $\sum d_{ap}$ in Equation 17-6).

Exhibit 17-13
Delay due to Turning Vehicles

Midsegment Volume (veh/h/ln)	Through Vehicle Delay (s/veh/pt) by Number of Through Lanes		
	1 Lane	2 Lanes	3 Lanes
200	0.04	0.04	0.05
300	0.08	0.08	0.09
400	0.12	0.15	0.15
500	0.18	0.25	0.15
600	0.27	0.41	0.15
700	0.39	0.72	0.15

The values listed in Exhibit 17-13 represent 10% left turns and 10% right turns from the segment at the access point intersection. If the actual turn percentages are less than 10%, then the delays can be reduced proportionally. For example, if the subject access point has 5% left turns and 5% right turns, then the values listed in the exhibit should be multiplied by 0.5 (= 5/10). Also, if a turn bay

of adequate length is provided for one turn movement but not the other, then the values listed in the exhibit should be multiplied by 0.5. If both turn movements are provided a bay of adequate length, then the delay due to turns can be assumed to equal 0.0 seconds per vehicle per access point (s/veh/pt).

D. Estimate Delay due to Other Sources

Numerous other factors could cause a driver to reduce speed or to incur delay while traveling along a segment. For example, a vehicle that is completing a parallel parking maneuver may cause following vehicles to incur some delay. Also, vehicles that yield to pedestrians at a midsegment crosswalk may incur delay. Finally, bicyclists riding in a traffic lane or an adjacent bicycle lane may directly or indirectly cause vehicular traffic to adopt a lower speed.

Of the many sources for midsegment delay, the automobile methodology only includes procedures for estimating the delay due to turning vehicles. However, if the delay due to other sources is known or estimated by other means, then it can be included in the equation to compute running time.

E. Compute Segment Running Time

Equation 17-6 is used to compute segment running time based on consideration of through movement control at the boundary intersection, free-flow speed, vehicle proximity, and various midsegment delay sources.

Equation 17-6

$$t_R = \frac{6.0 - l_1}{0.0025\,L} f_x + \frac{3{,}600\,L}{5{,}280\,S_f} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{\text{other}}$$

with

Equation 17-7

$$f_x = \begin{cases} 1.00 & \text{(signalized or STOP-controlled through movement)} \\ 0.00 & \text{(uncontrolled through movement)} \\ \min[v_{th}/c_{th},\, 1.00] & \text{(YIELD-controlled through movement)} \end{cases}$$

where

t_R = segment running time (s);

l_1 = start-up lost time = 2.0 if signalized, 2.5 if STOP or YIELD controlled (s);

L = segment length (ft);

f_x = control-type adjustment factor;

v_{th} = through-demand flow rate (veh/h);

c_{th} = through-movement capacity (veh/h);

$d_{ap,i}$ = delay due to left and right turns from the street into access point intersection i (s/veh);

N_{ap} = number of influential access point approaches along the segment = $N_{ap,s} + p_{ap,lt} N_{ap,o}$ (points);

$N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points);

$N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points);

$p_{ap,lt}$ = proportion of $N_{ap,o}$ that can be accessed by a left turn from the subject direction of travel; and

d_{other} = delay due to other sources along the segment (e.g., curb parking or pedestrians) (s/veh).

Other variables are as previously defined. The variables l_1, f_x, v_{th}, and c_{th} used with the first term in Equation 17-6 apply to the through movement exiting the segment at the boundary intersection. This term accounts for the time required to accelerate to the running speed, less the start-up lost time. The divisor in this term is an empirical adjustment that minimizes the contribution of this term for longer segments. It partially reflects a tendency for drivers to offset this added time by adopting slightly higher midsegment speeds than reflected in the start-up lost time estimate.

Step 3: Determine the Proportion Arriving During Green

This step applies to the downstream boundary intersection when the operation of a signalized urban street segment is evaluated. If the downstream boundary intersection is not signalized, then this step is skipped.

The methodology includes a procedure for computing the proportion of vehicles that arrive during the effective green time for a phase serving a segment lane group (i.e., the lane groups "internal" to the segment). This procedure is described in this step. The procedure described in Chapter 18, Signalized Intersections, should be used for phases serving external lane groups.

If the upstream intersection is not signalized (or it is signalized but not coordinated with the downstream boundary intersection), then the proportion arriving during the green is equal to the effective green-to-cycle-length ratio and this step is completed. This relationship implies that arrivals are effectively uniform during the cycle when averaged over the analysis period.

If the boundary intersections are coordinated, then the remaining discussion in this step applies. The calculation of the proportion arriving during green is based on the signal timing of the upstream and downstream boundary intersections. However, if the signals are actuated, then the resulting estimate of the proportion arriving during green typically has an effect on signal timing and capacity. In fact, the process is circular and requires an iterative sequence of calculations to arrive at a convergence solution in which all computed variables are in agreement with their initially assumed values. This process is illustrated in Exhibit 17-8. This exhibit indicates that the calculation of average phase duration is added to this process when the intersection is actuated.

Typically, there are three signalized traffic movements that depart the upstream boundary intersection at different times during the signal cycle. They are the cross-street right turn, major-street through, and cross-street left turn. Traffic may also enter the segment at various access point intersections. The signalized movements often enter the segment as a platoon, but this platoon disperses as the vehicles move down the segment.

A platoon dispersion model is used to predict the dispersed flow rate as a function of running time at any specified downstream location. The dispersed flow rates for the upstream intersection movement are combined with access point flow rates to predict an arrival flow profile at the downstream location. Exhibit 17-14 illustrates the predicted arrival flow profile at the stop line of the downstream intersection. This profile reflects the combination of the left-turn, through, and right-turn movements from the upstream intersection plus the turn movements at the access point intersection. The platoon dispersion model and the manner in which it is used to predict the dispersed flow rates for each of the individual movements are described in Chapter 30.

The gray shaded area in Exhibit 17-14 represents the arrival count during green n_g. This count is computed by summing the flow rate for each time "step" (or interval) that occurs during the effective green period. The proportion of vehicles arriving during the effective green period for a specified lane group is computed by using Equation 17-8.

Equation 17-8

$$P = \frac{n_g}{q_d\, C}$$

where

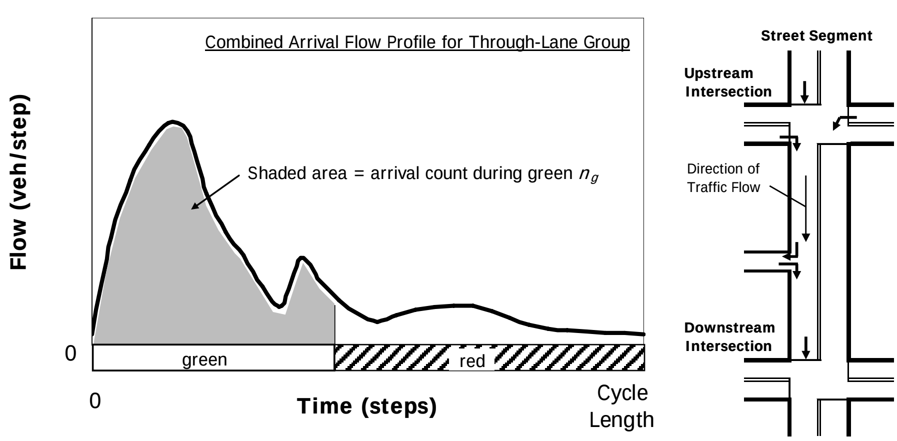
P = proportion of vehicles arriving during the green indication,

n_g = arrival count during green (veh),

q_d = arrival flow rate for downstream lane group (veh/s), and

C = cycle length (s).

Exhibit 17-14
Use of an Arrival Flow Profile to Estimate the Volume Arriving During Green



Step 4: Determine Signal Phase Duration

This step applies to the downstream boundary intersection when the operation of a signalized urban street segment is evaluated. If the downstream boundary intersection is not signalized, then this step is skipped.

If the downstream boundary intersection has pretimed signal control, then the signal phase duration is an input value. If this intersection has some form of actuated control, then the procedure described in Chapter 18 is used to estimate the average phase duration.

Steps 1 to 4 are repeated until the duration of each phase at each signalized intersection converges to its steady-state value. Convergence is indicated when the estimate of phase duration on two successive repetitions is the same.

Step 5: Determine Through Delay

The delay incurred by through vehicles as they exit the segment is the basis for travel time estimation. In this context, a through vehicle is a vehicle that enters and exits the segment as a through vehicle. The nature of the delay models used in this manual makes it difficult to separate the delay to through vehicles from the delay to nonthrough vehicles. However, these models can provide a reasonable estimate of through delay whenever the through movement is the dominant movement on the segment.

Through delay represents the sum of two delay sources. One source is the delay due to the traffic control at the boundary intersection. It is called control delay. The other delay is that due to the negotiation of intersection geometry, such as curvature. It is called geometric delay.

Procedures for computing control delay are described in the following chapters of this manual:

- Signal control (Chapter 18 or 22),
- All-way STOP control (Chapter 20), and
- YIELD control at a roundabout intersection (Chapter 21).

The analyst should refer to the appropriate chapter for guidance in estimating the through control delay for the boundary intersection. If the through movement is uncontrolled at the boundary intersection, then the through control delay is 0.0 s/veh.

The geometric delay for conventional three-leg or four-leg intersections (i.e., noncircular intersections) is considered to be negligible. In contrast, the geometric delay for a circular intersection is not negligible and should be added to the control delay to obtain the necessary through delay. A procedure for estimating geometric delay for roundabout intersections is described in Chapter 33, Roundabouts: Supplemental.

If the segment is not in a coordinated system, the through delay estimate should be based on isolated operation. The methodologies in Chapters 18 to 21 can be used to provide this estimate.

If the segment is within a coordinated signal system, then the methodology in Chapter 18 or Chapter 22 is used to determine the through delay. The upstream filtering adjustment factor is used to account for the effect of the upstream signal on the variability in arrival volume at the downstream intersection. The procedure for calculating this factor is described in Section 1 of Chapter 18.

If the through movement shares one or more lanes at a signalized boundary intersection, then the through delay is computed by using Equation 17-9.

Equation 17-9

$$d_t = \frac{d_{th}\ v_t\ N_t + d_{sl}\ v_{sl}(1-P_L) + d_{sr}\ v_{sr}(1-P_R)}{v_{th}}$$

where

d_t = through delay (s/veh),

v_{th} = through-demand flow rate (veh/h),

d_{th} = delay in exclusive through-lane group (s/veh),

v_t = demand flow rate in exclusive through-lane group (veh/h/ln),

N_t = number of lanes in exclusive through-lane group (ln),

d_{sl} = delay in shared left-turn and through-lane group (s/veh),

v_{sl} = demand flow rate in shared left-turn and through-lane group (veh/h),

d_{sr} = delay in shared right-turn and through-lane group (s/veh),

v_{sr} = demand flow rate in shared right-turn and through-lane group (veh/h),

P_L = proportion of left-turning vehicles in the shared lane (decimal), and

P_R = proportion of right-turning vehicles in the shared lane (decimal).

The procedure described in Chapter 18, Signalized Intersections, is used to estimate the variables shown in Equation 17-9.

Step 6: Determine Through Stop Rate

As with control delay, through stop rate describes the stop rate of vehicles that enter and exit the segment as through vehicles. The nature of the stop rate models described in this step makes it difficult to separate the stops to through vehicles from those incurred by nonthrough vehicles. However, these models can provide a reasonable estimate of through stop rate whenever the through movement is the dominant movement on the segment.

Stop rate is defined as the average number of full stops per vehicle. A *full stop* is defined to occur at a signalized intersection when a vehicle slows to zero (or a crawl speed, if in queue) as a consequence of the change in signal indication from green to red, but not necessarily in direct response to an observed red indication. A *full stop* is defined to occur at an unsignalized intersection when a vehicle slows to zero (or a crawl speed, if in queue) as a consequence of the control device used to regulate the approach. For example, if a vehicle is in an overflow queue and requires three signal cycles to clear the intersection, then it is estimated to have three full stops (one stop for each cycle).

The stop rate for a STOP-controlled approach can be assumed to equal 1.0 stops/veh. The stop rate for an uncontrolled approach can be assumed to equal 0.0 stops/veh. The stop rate at a YIELD-controlled approach will vary with conflicting demand. It can be estimated (in stops per vehicle) as equal to the volume-to-capacity ratio of the through movement at the boundary intersection. This approach recognizes that YIELD control does not require drivers to come to a complete stop when there is no conflicting traffic.

The through stop rate at a signalized boundary intersection is computed by using Equation 17-10.

$$h = 3{,}600\left(\frac{N_f}{\min\left(1, \frac{v_{th}\,C}{N_{th}\,s\,g}\right) g\,s} + \frac{N_{th}\,Q_{2+3}}{v_{th}\,C}\right)$$

Equation 17-10

with

$$N_f = \frac{N_{f,t}\,N_t + N_{f,sl}\,(1 - P_L) + N_{f,sr}\,(1 - P_R)}{N_{th}}$$

Equation 17-11

$$s = \frac{s_t N_t + s_{sl}(1 - P_L) + s_{sr}(1 - P_R)}{N_{th}}$$

Equation 17-12

$$Q_{2+3} = \frac{(Q_{2,t} + Q_{3,t})\,N_t + (Q_{2,sl} + Q_{3,sl})\,(1 - P_L) + (Q_{2,sr} + Q_{3,sr})\,(1 - P_R)}{N_{th}}$$

Equation 17-13

where

h = full stop rate (stops/veh),

N_f = number of fully stopped vehicles (veh/ln),

g = effective green time (s),

s = adjusted saturation flow rate (veh/h/ln),

Q_{2+3} = back-of-queue size (veh/ln),

$N_{f,t}$ = number of fully stopped vehicles in exclusive through-lane group (veh/ln),

$N_{f,sl}$ = number of fully stopped vehicles in shared left-turn and through-lane group (veh/ln),

$N_{f,sr}$ = number of fully stopped vehicles in shared right-turn and through-lane group (veh/ln),

N_{th} = number of through lanes (shared or exclusive) (ln),

s_t = saturation flow rate in exclusive through-lane group (veh/h/ln),

s_{sl} = saturation flow rate in shared left-turn and through-lane group with permitted operation (veh/h/ln),

s_{sr} = saturation flow rate in shared right-turn and through-lane group with permitted operation (veh/h/ln),

$Q_{2,t}$ = second-term back-of-queue size for exclusive through-lane group (veh/ln),

$Q_{2,sl}$ = second-term back-of-queue size for shared left-turn and through-lane group (veh/ln),

$Q_{2,sr}$ = second-term back-of-queue size for shared right-turn and through-lane group (veh/ln),

$Q_{3,t}$ = third-term back-of-queue size for exclusive through-lane group (veh/ln),

$Q_{3,sl}$ = third-term back-of-queue size for shared left-turn and through-lane group (veh/ln), and

$Q_{3,sr}$ = third-term back-of-queue size for shared right-turn and through-lane group (veh/ln).

Other variables are as previously defined. The procedure for computing N_f, Q_2, and Q_3 is provided in Chapter 31, Signalized Intersections: Supplemental.

The first term in Equation 17-10 represents the proportion of vehicles stopped once by the signal. For some of the more complex arrival–departure polygons that include left-turn movements operating with the permitted mode, the queue may dissipate at two or more points during the cycle. If this occurs, then $N_{f,i}$ is computed for each of the i periods between queue dissipation points. The value of N_f then equals the sum of the $N_{f,i}$ values computed in this manner.

The second term in Equation 17-10 represents the additional stops that may occur during overflow (i.e., cycle failure) conditions. The contribution of this term becomes significant when the volume-to-capacity ratio exceeds about 0.8. The full stop rate typically varies from 0.4 stops/veh at low volume-to-capacity ratios to 2.0 stops/veh when the volume-to-capacity ratio is about 1.0.

Step 7: Determine Travel Speed

Equation 17-14 is used to compute the travel speed for the subject direction of travel along the segment.

Equation 17-14

$$S_{T,seg} = \frac{3{,}600\ L}{5{,}280\ (t_R + d_t)}$$

where

$S_{T,seg}$ = travel speed of through vehicles for the segment (mi/h),

L = segment length (ft),

t_R = segment running time (s), and

d_t = through delay (s/veh).

The control delay used in Equation 17-14 is that incurred by the through-lane group at the downstream boundary intersection.

Step 8: Determine Spatial Stop Rate

Equation 17-15 is used to compute the spatial stop rate for the subject direction of travel along the segment.

Equation 17-15

$$H_{seg} = 5{,}280\ \frac{h + h_{\text{other}}}{L}$$

where

H_{seg} = spatial stop rate for the segment (stops/mi),

h = full stop rate (stops/veh),

h_{other} = full stop rate due to other sources (stops/veh), and

L = segment length (ft).

The full stop rate h used in Equation 17-15 is that incurred by the through-lane group at the downstream boundary intersection. In some situations, stops may be incurred at midsegment locations due to pedestrian crosswalks, bus stops, or turns into access point approaches. If the full stop rate associated with these other stops can be estimated by the analyst, then it can be included in the calculation by using the variable h_{other}.

Step 9: Determine LOS

LOS is determined for both directions of travel along the segment. Exhibit 17-2 lists the LOS thresholds established for this purpose. As indicated in this exhibit, LOS is defined by two performance measures. One measure is the travel speed for through vehicles, expressed as a percentage of the base free-flow speed. The second measure is the volume-to-capacity ratio for the through movement at the downstream boundary intersection.

The base free-flow speed was computed in Step 2 and the travel speed was computed in Step 7.

The volume-to-capacity ratio for the through movement at the boundary intersection is computed as the through volume divided by the through-movement capacity. This capacity is an input variable to the methodology.

The LOS attributed to each direction of travel applies to the segment, which includes both the link and the downstream boundary intersection. Chapters 18 to 22 describe LOS thresholds for the boundary intersection. The automobile methodology does not assign a LOS indicator to the link portion of the segment.

LOS is probably more meaningful as an indicator of traffic performance along a facility rather than a single street segment. A procedure for estimating facility LOS is described in Chapter 16.

Step 10: Determine Automobile Traveler Perception Score

The automobile traveler perception score for urban street segments is provided as a useful performance measure. It indicates the traveler's perception of service quality. The score is computed by using Equation 17-16 to Equation 17-21.

$$I_{a,seg} = 1 + P_{BCDEF} + P_{CDEF} + P_{DEF} + P_{EF} + P_F$$

Equation 17-16

with

$$P_{BCDEF} = \left(1 + e^{-1.1614 - 0.253\,H_{seg} + 0.3434\,P_{LTL,seg}}\right)^{-1}$$

Equation 17-17

$$P_{CDEF} = \left(1 + e^{0.6234 - 0.253\,H_{seg} + 0.3434\,P_{LTL,seg}}\right)^{-1}$$

Equation 17-18

$$P_{DEF} = \left(1 + e^{1.7389 - 0.253\,H_{seg} + 0.3434\,P_{LTL,seg}}\right)^{-1}$$

Equation 17-19

$$P_{EF} = \left(1 + e^{2.7047 - 0.253\,H_{seg} + 0.3434\,P_{LTL,seg}}\right)^{-1}$$

Equation 17-20

Equation 17-21

$$P_F = \left(1 + e^{3.8044 - 0.253 H_{seg} + 0.3434 P_{LTL,seg}}\right)^{-1}$$

where

$I_{a,seg}$ = automobile traveler perception score for segment;

P_{BCDEF} = probability that an individual will respond with a rating of B, C, D, E, or F;

P_{CDEF} = probability that an individual will respond with a rating of C, D, E, or F;

P_{DEF} = probability that an individual will respond with a rating of D, E, or F;

P_{EF} = probability that an individual will respond with a rating of E or F;

P_F = probability that an individual will respond with a rating of F; and

$P_{LTL,seg}$ = proportion of intersections with a left-turn lane (or bay) on the segment (decimal).

Other variables are as previously defined. The derivation of Equation 17-16 is based on the assignment of scores to each letter rating, in which a score of "1" is assigned to the rating of A (denoting "best"), "2" is assigned to B, and so on. The survey results were used to calibrate a set of models that collectively predicts the probability that a traveler will assign various rating combinations for a specified spatial stop rate and proportion of intersections with left-turn lanes. The score obtained from Equation 17-16 represents the expected (or long-run average) score for the population of travelers.

The proportion of intersections with left-turn lanes equals the number of left-turn lanes (or bays) encountered while driving along the segment divided by the number of intersections encountered. The signalized boundary intersection is counted (if it exists). All unsignalized intersections of public roads are counted. Private driveway intersections are not counted, unless they are signal controlled.

The score obtained from Equation 17-16 provides a useful indication of performance from the perspective of the traveler. Scores of 2.0 or less indicate the best perceived service, and values in excess of 5.0 indicate the worst perceived service. Although this score is closely tied to the concept of service quality, it is *not* used to determine LOS for the urban street segment.

PEDESTRIAN MODE

This subsection describes the methodology for evaluating the performance of an urban street segment in terms of its service to pedestrians.

Urban street segment performance from a pedestrian perspective is separately evaluated for each side of the street. *Unless otherwise stated, all variables identified in this section are specific to the subject side of the street.* If a sidewalk is not available for the subject side of the street, then it is assumed that pedestrians will walk in the street on that side (even if there is a sidewalk on the other side).

The methodology is focused on the analysis of a segment with either signal-controlled or two-way STOP-controlled boundary intersections. Chapter 18 describes a methodology for evaluating signalized intersection performance from

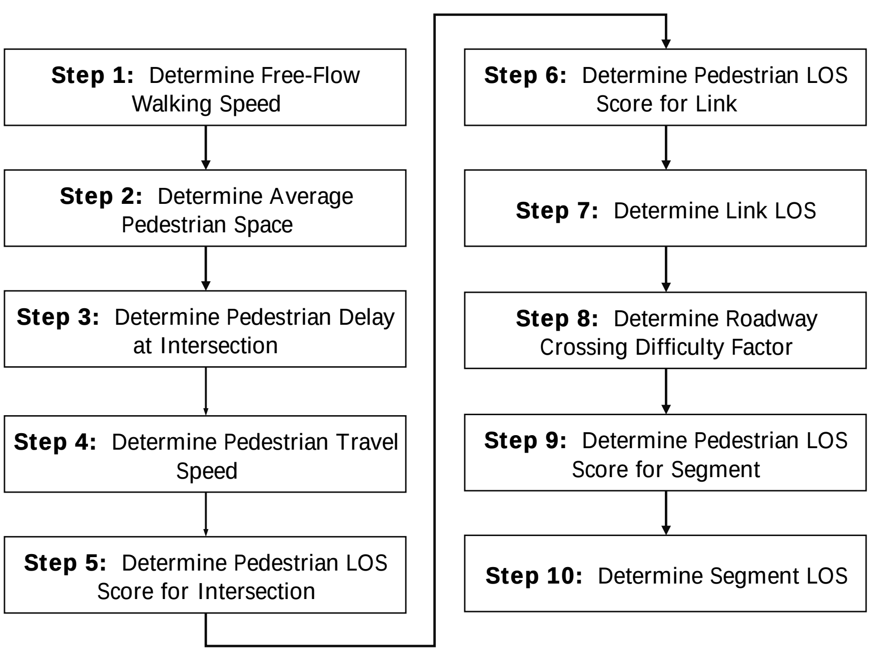
a pedestrian perspective. No methodology exists for evaluating two-way STOP-controlled intersection performance (with the cross street STOP controlled). However, it is reasoned that this type of control has negligible influence on pedestrian service along the segment. This edition of the HCM does not include a procedure for evaluating a segment's performance when the boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

The pedestrian methodology is applied through a series of nine steps that culminate in the determination of the segment LOS. These steps are illustrated in Exhibit 17-15. Performance measures that are estimated include

- Pedestrian travel speed,
- Average pedestrian space, and
- Pedestrian LOS scores for the link and segment.

A methodology for evaluating off-street pedestrian facilities is provided in Chapter 23, Off-Street Pedestrian and Bicycle Facilities.

Exhibit 17-15
Pedestrian Methodology for Urban Street Segments



Link-Based Evaluation

Steps 6 and 7 of the pedestrian methodology can be used as a stand-alone procedure for link-based evaluation of pedestrian service. This approach is regularly used by local, regional, and state transportation agencies. It offers the advantage of being less data-intensive than the full, 10-step methodology and produces results that are generally reflective of pedestrian perceptions of service along the roadway. It can be especially attractive when agencies are performing a networkwide evaluation for a large number of roadway links.

The analyst should recognize that the resulting link LOS does not consider some aspects of pedestrian travel along a segment (e.g., crossing difficulty or intersection service). For this reason, the LOS score for the link should not be

aggregated for the purpose of characterizing facility performance. The analyst should also be aware that this approach precludes an integrated multimodal evaluation because it does not fully reflect segment performance.

Concepts

The methodology provides a variety of measures for evaluating segment performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip along the segment. One measure is the LOS score. This score is an indication of the typical pedestrian's perception of the overall segment travel experience. A second measure is the average speed of pedestrians traveling along the segment.

A third measure is based on the concept of "circulation area." It represents the average amount of sidewalk area available to each pedestrian walking along the segment. A larger area is more desirable from the pedestrian perspective. Exhibit 17-16 provides a qualitative description of pedestrian space that can be used to evaluate sidewalk performance from a circulation-area perspective.

Exhibit 17-16
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p)		
Random Flow	Platoon Flow	Description
>60	>530	Ability to move in desired path, no need to alter movements
>40–60	>90–530	Occasional need to adjust path to avoid conflicts
>24–40	>40–90	Frequent need to adjust path to avoid conflicts
>15–24	>23–40	Speed and ability to pass slower pedestrians restricted
>8–15	>11–23	Speed restricted, very limited ability to pass slower pedestrians
≤8	≤11	Speed severely restricted, frequent contact with other users

The first two columns in Exhibit 17-16 indicate a sensitivity to flow condition. Random pedestrian flow is typical of most segments. Platoon flow is appropriate for shorter segments (e.g., in downtown areas) with signalized boundary intersections.

Step 1: Determine Free-Flow Walking Speed

The *average* free-flow pedestrian walking speed S_{pf} is needed for the evaluation of urban street segment performance from a pedestrian perspective. This speed should reflect conditions in which there are negligible pedestrian-to-pedestrian conflicts and negligible adjustments in a pedestrian's desired walking path to avoid other pedestrians.

Research indicates that walking speed is influenced by pedestrian age and sidewalk grade (*6*). If 0% to 20% of pedestrians traveling along the subject segment are elderly (i.e., 65 years of age or older), an average free-flow walking speed of 4.4 ft/s is recommended for segment evaluation. If more than 20% of pedestrians are elderly, an average free-flow walking speed of 3.3 ft/s is recommended. In addition, an upgrade of 10% or greater reduces walking speed by 0.3 ft/s.

Step 2: Determine Average Pedestrian Space

Pedestrians are sensitive to the amount of space separating them from other pedestrians and obstacles as they walk along a sidewalk. Average pedestrian

space is an indicator of segment performance for travel in a sidewalk. It depends on the effective sidewalk width, pedestrian flow rate, and walking speed. This step is not applicable when the sidewalk does not exist.

A. Compute Effective Sidewalk Width

The effective sidewalk width equals the total walkway width less the effective width of fixed objects located on the sidewalk and less any shy distance associated with the adjacent street or a vertical obstruction. Fixed objects can be continuous (e.g., a fence or a building face) or discontinuous (e.g., trees, poles, or benches).

The effective sidewalk width is an average value for the length of the link. It is computed by using Equation 17-22 to Equation 17-26.

$$W_E = W_T - W_{O,i} - W_{O,o} - W_{s,i} - W_{s,o} \quad \geq 0.0$$

Equation 17-22

with

$$W_{s,i} = \max(W_{buf}, 1.5)$$

Equation 17-23

$$W_{s,o} = 3.0 p_{\text{window}} + 2.0\, p_{\text{building}} + 1.5\, p_{\text{fence}}$$

Equation 17-24

$$W_{O,i} = w_{O,i} - W_{s,i} \quad \geq 0.0$$

Equation 17-25

$$W_{O,o} = w_{O,o} - W_{s,o} \quad \geq 0.0$$

Equation 17-26

where

W_E = effective sidewalk width (ft),

W_T = total walkway width (ft),

$W_{O,i}$ = adjusted fixed-object effective width on inside of sidewalk (ft),

$W_{O,o}$ = adjusted fixed-object effective width on outside of sidewalk (ft),

$W_{s,i}$ = shy distance on inside (curb side) of sidewalk (ft),

$W_{s,o}$ = shy distance on outside of sidewalk (ft),

W_{buf} = buffer width between roadway and sidewalk (ft),

p_{window} = proportion of sidewalk length adjacent to a window display (decimal),

p_{building} = proportion of sidewalk length adjacent to a building face (decimal),

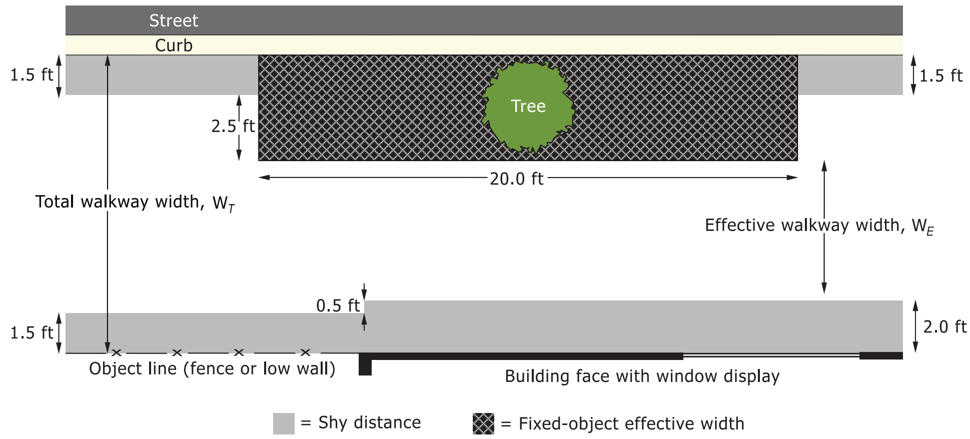
p_{fence} = proportion of sidewalk length adjacent to a fence or low wall (decimal),

$w_{O,i}$ = effective width of fixed objects on inside of sidewalk (ft), and

$w_{O,o}$ = effective width of fixed objects on outside of sidewalk (ft).

The relationship between the variables in these equations is illustrated in Exhibit 17-17.

Exhibit 17-17
Width Adjustments for Fixed Objects



The variables W_T, W_{buf}, p_{window}, p_{building}, p_{fence}, $w_{O,i}$, and $w_{O,o}$ are input variables. They represent average, or typical, values for the length of the sidewalk. Chapter 23, Off-Street Pedestrian and Bicycle Facilities, provides guidance for estimating the effective width of many common fixed objects.

Typical shy distances are shown in Exhibit 17-17. Shy distance on the inside (curb side) of the sidewalk is measured from the outside edge of the paved roadway (or face of curb, if present). It is generally considered to equal 1.5 ft. Shy distance on the outside of the sidewalk is 1.5 ft if a fence or a low wall is present, 2.0 ft if a building is present, 3.0 ft if window display is present, and 0.0 ft otherwise.

B. Compute Pedestrian Flow Rate per Unit Width

The pedestrian flow per unit width of sidewalk is computed by using Equation 17-27 for the subject sidewalk. The variable v_{ped} is an input variable.

Equation 17-27

$$v_p = \frac{v_{ped}}{60\ W_E}$$

where

v_p = pedestrian flow per unit width (p/ft/min),

v_{ped} = pedestrian flow rate in the subject sidewalk (walking in both directions) (p/h), and

W_E = effective sidewalk width (ft).

C. Compute Average Walking Speed

The average walking speed S_p is computed by using Equation 17-28. This equation is derived from the relationship between flow rate and average walking speed described in Exhibit 23-1 of Chapter 23.

Equation 17-28

$$S_p = (1 - 0.00078\ v_p^2)\ S_{pf} \quad \geq 0.5\ S_{pf}$$

where S_p = pedestrian walking speed (ft/s), S_{pf} = free-flow pedestrian walking speed (ft/s), and v_p = pedestrian flow per unit width (p/ft/min).

D. Compute Pedestrian Space

Finally, Equation 17-29 is used to compute average pedestrian space.

$$A_p = 60 \frac{S_p}{v_p}$$

Equation 17-29

where A_p is the pedestrian space (ft²/p) and other variables are as previously defined.

The pedestrian space obtained from Equation 17-29 can be compared with the ranges provided in Exhibit 17-16 to make some judgments about the performance of the subject intersection corner.

Step 3: Determine Pedestrian Delay at Intersection

Pedestrian delay at three locations along the segment is determined in this step. Each of these delays represents an input variable for the methodology and is described in Section 1, Required Input Data.

The first delay variable represents the delay incurred by pedestrians who travel through the boundary intersection along a path that is parallel to the segment centerline d_{pp}. The second delay variable represents the delay incurred by pedestrians who cross the segment at the nearest signal-controlled crossing d_{pc}. The third delay variable represents the delay incurred by pedestrians waiting for a gap to cross the segment at an uncontrolled location d_{pw}.

Step 4: Determine Pedestrian Travel Speed

Pedestrian travel speed represents an aggregate measure of speed along the segment. It combines the delay incurred at the downstream boundary intersection plus the time required to walk the length of the segment. As such, it is typically slower than the average walking speed. The pedestrian travel speed is computed by using Equation 17-30.

$$S_{Tp,seg} = \frac{L}{\frac{L}{S_p} + d_{pp}}$$

Equation 17-30

where

$S_{Tp,seg}$ = travel speed of through pedestrians for the segment (ft/s),

L = segment length (ft),

S_p = pedestrian walking speed (ft/s), and

d_{pp} = pedestrian delay when walking parallel to the segment (s/p).

In general, a travel speed of 4.0 ft/s or more is considered desirable and a speed of 2.0 ft/s or less is considered undesirable.

Step 5: Determine Pedestrian LOS Score for Intersection

The pedestrian LOS score for the boundary intersection $I_{p,int}$ is determined in this step. If the boundary intersection is signalized, then the pedestrian

methodology described in Chapter 18 is used for this determination. If the boundary intersection is two-way STOP controlled, then the score is equal to 0.0.

Step 6: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link $I_{p,link}$ is calculated by using Equation 17-31.

Equation 17-31
$$I_{p,link} = 6.0468 + F_w + F_v + F_S$$

with

Equation 17-32
$$F_w = -1.2276\ \ln(W_v + 0.5\ W_1 + 50\ p_{pk} + W_{buf}\ f_b + W_{aA}\ f_{sw})$$

Equation 17-33
$$F_v = 0.0091\ \frac{v_m}{4\ N_{th}}$$

Equation 17-34
$$F_s = 4\left(\frac{S_R}{100}\right)^2$$

where

$I_{p,link}$ = pedestrian LOS score for link;

F_w = cross-section adjustment factor;

F_v = motorized vehicle volume adjustment factor;

F_S = motorized vehicle speed adjustment factor;

$\ln(x)$ = natural log of x;

W_v = effective total width of outside through lane, bicycle lane, and shoulder as a function of traffic volume (see Exhibit 17-18) (ft);

W_1 = effective width of combined bicycle lane and shoulder (see Exhibit 17-18) (ft);

p_{pk} = proportion of on-street parking occupied (decimal);

W_{buf} = buffer width between roadway and available sidewalk (= 0.0 if sidewalk does not exist) (ft);

f_b = buffer area coefficient = 5.37 for any continuous barrier at least 3 ft high that is located between the sidewalk and the outside edge of roadway; otherwise use 1.0;

W_A = available sidewalk width = 0.0 if sidewalk does not exist or $W_T - W_{buf}$ if sidewalk exists (ft);

W_{aA} = adjusted available sidewalk width = min(W_A, 10) (ft);

f_{sw} = sidewalk width coefficient = 6.0 – 0.3 W_{aA};

v_m = midsegment demand flow rate (direction nearest to the subject sidewalk) (veh/h);

N_{th} = number of through lanes on the segment in the subject direction of travel (ln); and

S_R = motorized vehicle running speed = (3,600 L)/(5,280 t_R) (mi/h).

The value used for several of the variables in Equation 17-32 to Equation 17-34 is dependent on various conditions. These conditions are identified in Column 1 of Exhibit 17-18. If the condition is satisfied, then the equation in Column 2 is used to compute the variable value. If it is not satisfied, then the equation in Column 3 is used. The equations in the first two rows are considered in sequence to determine the effective width of the outside lane and shoulder W_v.

Exhibit 17-18
Variables for Pedestrian LOS Score for Link

Condition	Variable When Condition Is Satisfied	Variable When Condition Is Not Satisfied
p_{pk} = 0.0	$W_t = W_{ol} + W_{bl} + W_{os}^*$	$W_t = W_{ol} + W_{bl}$
v_m > 160 veh/h or street is divided	$W_v = W_t$	$W_v = W_t\,(2 - 0.005\ v_m)$
p_{pk} < 0.25 or parking is striped	$W_1 = W_{bl} + W_{os}^*$	$W_1 = 10$

Notes: W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);
W_{ol} = width of the outside through lane (ft);
W_{os}^* = adjusted width of paved outside shoulder; if curb is present $W_{os}^* = W_{os} - 1.5 \geq 0.0$, otherwise $W_{os}^* = W_{os}$ (ft);
W_{os} = width of paved outside shoulder (ft); and
W_{bl} = width of the bicycle lane = 0.0 if bicycle lane not provided (ft).

The buffer width coefficient determination is based on the presence of a continuous barrier in the buffer. In making this determination, repetitive vertical objects (e.g., trees or bollards) are considered to represent a continuous barrier if they are at least 3 ft high and have an average spacing of 20 ft or less. For example, the sidewalk shown in Exhibit 17-17 does not have a continuous buffer because the street trees adjacent to the curb are spaced at more than 20 ft.

The pedestrian LOS score is sensitive to the separation between pedestrians and moving vehicles; it is also sensitive to the speed and volume of these vehicles. Physical barriers and parked cars between moving vehicles and pedestrians effectively increase the separation distance and the perceived quality of service. Higher vehicle speeds or volumes lower the perceived quality of service.

If the sidewalk is not continuous for the length of the segment, then the segment should be subdivided into subsegments and each subsegment separately evaluated. For this application, a subsegment is defined to begin or end at each break in the sidewalk. Each subsegment is then separately evaluated by using Equation 17-31. Each equation variable is uniquely quantified to represent the subsegment to which it applies. The buffer width and the effective sidewalk width are each set to 0.0 ft for any subsegment without a sidewalk. The pedestrian LOS score $I_{p,link}$ is then computed as a weighted average of the subsegment scores, where the weight assigned to each score equals the portion of the segment length represented by the corresponding subsegment.

The motorized vehicle running speed is computed by using the automobile methodology, as described in a previous subsection.

Step 7: Determine Link LOS

The pedestrian LOS for the link is determined by using the pedestrian LOS score from Step 6 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit

17-3 to determine the LOS for the specified direction of travel along the subject link. If a sidewalk does not exist and pedestrians are relegated to walking in the street, then LOS is determined by using Exhibit 17-4 because the pedestrian space concept does not apply.

Step 8: Determine Roadway Crossing Difficulty Factor

The pedestrian roadway crossing difficulty factor measures the difficulty of crossing the street between boundary intersections. Segment performance from a pedestrian perspective is reduced if the crossing is perceived to be difficult.

The roadway crossing difficulty factor is based on the delay incurred by a pedestrian who crosses the subject segment. One crossing option the pedestrian may consider is to alter his or her travel path by diverting to the nearest signal-controlled crossing. This crossing location may be a midsegment signalized crosswalk or it may be a signalized intersection.

A second crossing option is to continue on the original travel path by completing a midsegment crossing at an uncontrolled location. If this type of crossing is legal along the subject segment, then the pedestrian crosses when there is an acceptable gap in the motorized vehicle stream.

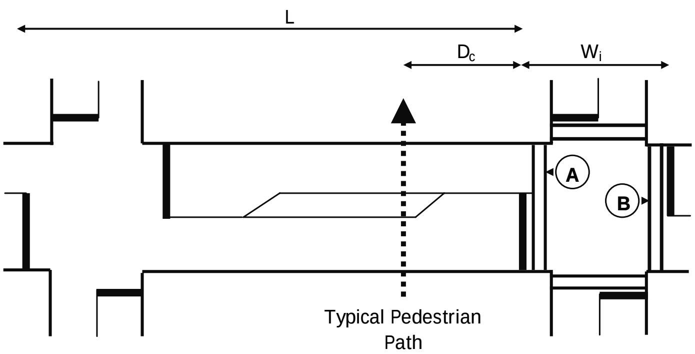
Each of these two crossing options is considered in this step, with that option requiring the least delay used as the basis for computing the pedestrian roadway crossing difficulty factor. The time to walk across the segment is common to both options and therefore is not included in the delay estimate for either option.

A. Compute Diversion Delay

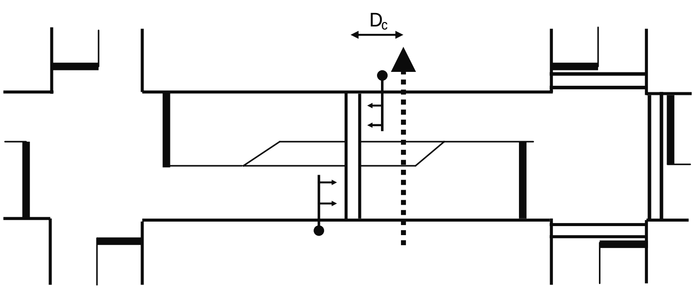
The delay incurred as a consequence of diverting to the nearest signal-controlled crossing is computed first. It includes the delay involved in walking to and from the midsegment crossing point to the nearest signal-controlled crossing and the delay waiting to cross at the signal. Hence, calculation of this delay requires knowledge of the distance to the nearest signalized crossing and its signal timing.

The distance to the nearest crossing location D_c is based on one of two approaches. The first approach is used if there is an identifiable pedestrian path (*a*) that intersects the segment and continues on beyond the segment and (*b*) on which most crossing pedestrians travel. The location of this path is shown for two cases in Exhibit 17-19. Exhibit 17-19(a) illustrates the distance D_c when the pedestrian diverts to the nearest signalized intersection. This distance is measured from the crossing location to the signalized intersection.

Exhibit 17-19
Diversion Distance Components



(a) Divert to Nearest Boundary Intersection



(b) Divert to Midsegment Signalized Crosswalk

Exhibit 17-19(b) illustrates the distance D_c when a signalized crosswalk is provided at a midsegment location. In this situation, the distance is measured from the pedestrian crossing location to the location of the signalized crosswalk. In either case, the distance D_c is an input value provided by the analyst.

The second approach is used if crossings occur somewhat uniformly along the length of the segment. In this situation the distance D_c can be assumed to equal one-third of the distance between the nearest signal-controlled crossings that bracket the subject segment.

The diversion distance to the nearest crossing is computed by using Equation 17-35.

Equation 17-35

$$D_d = 2\ D_c$$

where

D_d = diversion distance (ft), and

D_c = distance to nearest signal-controlled crossing (ft).

If the nearest crossing location is at the signalized intersection and the crossing is at Location A in Exhibit 17-19(a), then Equation 17-35 applies directly. If the nearest crossing location is at the signalized intersection but the crossing is at Location B, then the distance obtained from Equation 17-35 should be increased by adding two increments of the intersection width W_i.

The delay incurred due to diversion is calculated by using Equation 17-36.

Equation 17-36

$$d_{pd} = \frac{D_d}{S_p} + d_{pc}$$

where

d_{pd} = pedestrian diversion delay (s/p),

D_d = diversion distance (ft),

S_p = pedestrian walking speed (ft/s), and

d_{pc} = pedestrian delay when crossing the segment at the nearest signal-controlled crossing (s/p).

The pedestrian delay incurred when crossing at the nearest signal-controlled crossing was determined in Step 3.

B. Compute Roadway Crossing Difficulty Factor

The roadway crossing difficulty factor is computed by using Equation 17-37.

Equation 17-37

$$F_{cd} = 1.0 + \frac{0.10\ d_{px} - (0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606)}{7.5}$$

where

F_{cd} = roadway crossing difficulty factor,

d_{px} = crossing delay = min(d_{pd}, d_{pw}, 60) (s/p),

d_{pd} = pedestrian diversion delay (s/p),

d_{pw} = pedestrian waiting delay (s/p),

$I_{p,link}$ = pedestrian LOS score for link, and

$I_{p,int}$ = pedestrian LOS score for intersection.

If the factor obtained from Equation 17-37 is less than 0.80, then it is set equal to 0.80. If the factor is greater than 1.20, then it is set equal to 1.20.

The pedestrian waiting delay was determined in Step 3. If a midsegment crossing is illegal, then the crossing delay determination does not include consideration of the pedestrian waiting delay d_{pw} [i.e., d_{px} = min(d_{pd}, 60)].

Step 9: Determine Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is computed by using Equation 17-38.

Equation 17-38

$$I_{p,seg} = F_{cd}\left(0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606\right)$$

where $I_{p,seg}$ is the pedestrian LOS score for the segment and other variables are as previously defined.

Step 10: Determine Segment LOS

The pedestrian LOS for the segment is determined by using the pedestrian LOS score from Step 9 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit

17-3 to determine the LOS for the specified direction of travel along the subject segment. If a sidewalk does not exist and pedestrians are relegated to walking in the street, then LOS is determined by using Exhibit 17-4 because the pedestrian space concept does not apply.

BICYCLE MODE

This subsection describes the methodology for evaluating the performance of an urban street segment in terms of its service to bicyclists.

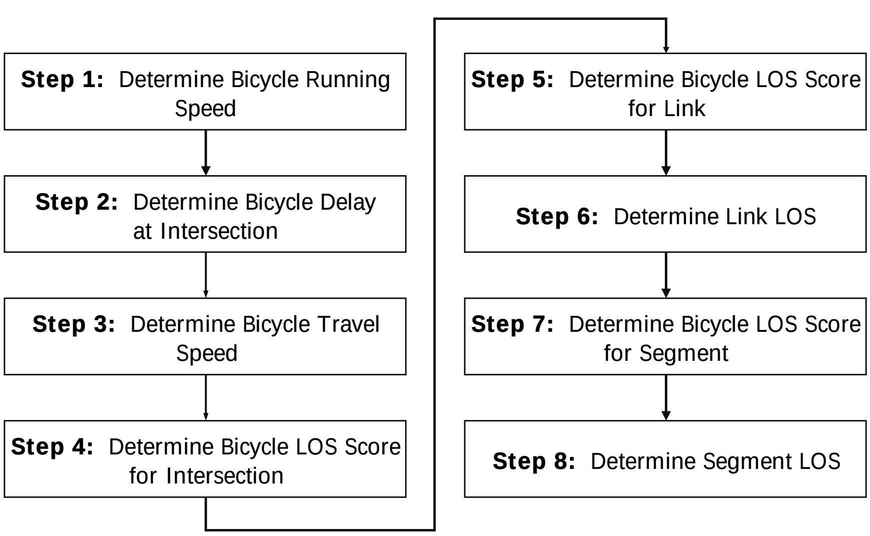
Urban street segment performance from a bicyclist perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.* The bicycle is assumed to travel in the street (possibly in a bicycle lane) and in the same direction as adjacent motorized vehicles.

The methodology is focused on the analysis of a segment with either signal-controlled or two-way STOP-controlled boundary intersections. Chapter 18 describes a methodology for evaluating signalized intersection performance from a bicyclist perspective. No methodology exists for evaluating two-way STOP-controlled intersection performance (with the cross street STOP controlled). However, the influence of this type of control is incorporated in the methodology for evaluating segment performance. This edition of the HCM does not include a procedure for evaluating a segment's performance when the boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

The bicycle methodology is applied through a series of seven steps that culminate in the determination of the segment LOS. These steps are illustrated in Exhibit 17-20. Performance measures that are estimated include bicycle travel speed and LOS scores for the link and segment.

A methodology for evaluating off-street bicycle facilities is provided in Chapter 23, Off-Street Pedestrian and Bicycle Facilities.

Exhibit 17-20
Bicycle Methodology for Urban Street Segments



Link-Based Evaluation

Steps 5 and 6 of the bicycle methodology can be used as a stand-alone procedure for link-based evaluation of bicycle service. This approach is regularly used by local, regional, and state transportation agencies. It offers the advantage of being less data-intensive than the full, eight-step methodology and produces results that are generally reflective of bicyclist perceptions of service along the roadway. It can be especially attractive when agencies are performing a networkwide evaluation for a large number of roadway links.

The analyst should recognize that the resulting link LOS does not consider some aspects of bicycle travel along a segment (e.g., intersection service). For this reason, the LOS score for the link should not be aggregated for the purpose of characterizing facility performance. The analyst should also be aware that this approach precludes an integrated multimodal evaluation because it does not fully reflect segment performance.

Step 1: Determine Bicycle Running Speed

An estimate of the *average* bicycle running speed S_b is determined in this step. The best basis for this estimate is a field measurement of midsegment bicycle speed on representative streets in the vicinity of the subject street. In the absence of this information, it is recommended that the average running speed of bicycles be taken as 15 mi/h between signalized intersections (*7*). It is recognized that many factors might affect bicycle speed, including adjacent motor vehicle traffic, adjacent on-street parking activity, commercial and residential driveways, lateral obstructions, and significant grades. To date, research is not available to make any specific recommendations as to the effect of these factors on speed.

Step 2: Determine Bicycle Delay at Intersection

Bicycle delay at the boundary intersection d_b is computed in this step. This delay is incurred by bicyclists who travel through the intersection in the same lane as (or in a bicycle lane that is parallel to the lanes used by) segment through vehicles.

If the boundary intersection is two-way STOP controlled (where the subject approach is uncontrolled), then the delay is equal to 0.0 s/bicycle. If the boundary intersection is signalized, then the delay is computed by using the methodology described in Chapter 18, Signalized Intersections.

Step 3: Determine Bicycle Travel Speed

Bicycle travel speed represents an aggregate measure of speed along the segment. It combines the delay incurred at the downstream boundary intersection and the time required to ride the length of the segment. As such, it is typically slower than the average bicycle running speed. The average bicycle travel speed is computed by using Equation 17-39.

Equation 17-39

$$S_{Tb,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rb} + d_b)}$$

where

$S_{Tb,seg}$ = travel speed of through bicycles along the segment (mi/h),

L = segment length (ft),

t_{Rb} = segment running time of through bicycles = (3,600 L)/(5,280 S_b) (s),

S_b = bicycle running speed (mi/h), and

d_b = bicycle control delay (s/bicycle).

In general, a travel speed of 10.0 mi/h or more is considered desirable and a speed of 5.0 mi/h or less is considered undesirable.

Step 4: Determine Bicycle LOS Score for Intersection

The bicycle LOS score for the boundary intersection $I_{b,int}$ is determined in this step. If the boundary intersection is signalized, then the bicycle methodology described in Chapter 18 is used for this determination. If the boundary intersection is two-way STOP controlled, then the score is equal to 0.0.

Step 5: Determine Bicycle LOS Score for Link

The bicycle LOS score for the segment $I_{b,link}$ is calculated by using Equation 17-40.

$$I_{b,link} = 0.760 + F_w + F_v + F_S + F_p$$

Equation 17-40

with

$$F_w = -0.005\ W_e^2$$

Equation 17-41

$$F_v = 0.507 \ln\left(\frac{v_{ma}}{4\ N_{th}}\right)$$

Equation 17-42

$$F_S = 0.199\ \left[1.1199 \ln(S_{Ra} - 20) + 0.8103\right]\left(1 + 0.1038\ \ P_{HVa}\right)^2$$

Equation 17-43

$$F_p = \frac{7.066}{P_c^{\ 2}}$$

Equation 17-44

where

$I_{b,link}$ = bicycle LOS score for link,

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_p = pavement condition adjustment factor,

$\ln(x)$ = natural log of x,

W_e = effective width of outside through lane (see Exhibit 17-21) (ft),

v_{ma} = adjusted midsegment demand flow rate (see Exhibit 17-21) (veh/h),

N_{th} = number of through lanes on the segment in the subject direction of travel (ln),

S_{Ra} = adjusted motorized vehicle running speed (see Exhibit 17-21) (mi/h),

P_{HVa} = adjusted percent heavy vehicles in midsegment demand flow rate (see Exhibit 17-21) (%), and

P_c = pavement condition rating (see Exhibit 17-7).

The value used for several of the variables in Equation 17-41 to Equation 17-44 is dependent on various conditions. These conditions are identified in Column 1 of Exhibit 17-21. If the condition is satisfied, then the equation in Column 2 is used to compute the variable value. If it is not satisfied, then the equation in Column 3 is used. The equations in the first three rows are considered in sequence to determine the effective width of the outside through lane W_e.

The motorized vehicle running speed is computed by using the automobile methodology described in a previous subsection.

Step 6: Determine Link LOS

The bicycle LOS for the link is determined by using the bicycle LOS score from Step 5. This performance measure is compared with the thresholds in Exhibit 17-4 to determine the LOS for the specified direction of travel along the subject link.

Exhibit 17-21
Variables for Bicycle LOS Score for Link

Condition	Variable When Condition Is Satisfied	Variable When Condition Is Not Satisfied
p_{pk} = 0.0	$W_t = W_{ol} + W_{bl} + W_{os}^*$	$W_t = W_{ol} + W_{bl}$
v_m > 160 veh/h or street is divided	$W_v = W_t$	$W_v = W_t (2 - 0.005\ v_m)$
$W_{bl} + W_{os}^*$ < 4.0 ft	$W_e = W_v - 10\ p_{pk} \geq 0.0$	$W_e = W_v + W_{bl} + W_{os}^* - 20\ p_{pk} \geq 0.0$
$v_m (1- 0.01\ P_{HV})$ < 200 veh/h and P_{HV} > 50%	P_{HVa} = 50%	$P_{HVa} = P_{HV}$
S_R < 21 mi/h	S_{Ra} = 21 mi/h	$S_{Ra} = S_R$
$v_m > 4\ N_{th}$	$v_{ma} = v_m$	$v_{ma} = 4\ N_{th}$

Notes:
W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);
W_{ol} = width of outside through lane (ft);
W_{os}^* = adjusted width of paved outside shoulder; if curb is present $W_{os}^* = W_{os} - 1.5 \geq 0.0$, otherwise $W_{os}^* = W_{os}$ (ft);
W_{os} = width of paved outside shoulder (ft);
W_{bl} = width of bicycle lane = 0.0 if bicycle lane not provided (ft);
W_v = effective total width of outside through lane, bicycle lane, and shoulder as a function of traffic volume (ft);
p_{pk} = proportion of on-street parking occupied (decimal);
v_m = midsegment demand flow rate (veh/h);
P_{HV} = percent heavy vehicles in the midsegment demand flow rate (%), and
S_R = motorized vehicle running speed (mi/h).

Step 7: Determine Bicycle LOS Score for Segment

The bicycle LOS score for the segment is computed by using Equation 17-45.

Equation 17-45

$$I_{b,seg} = 0.160\ I_{b,link} + 0.011\ F_{bi}\ e^{I_{b,int}} + 0.035\ \frac{N_{ap,s}}{(L/5280)} + 2.85$$

where

$I_{b,seg}$ = bicycle LOS score for segment;

$I_{b,link}$ = bicycle LOS score for link;

F_{bi} = indicator variable for boundary intersection control type = 1.0 if signalized, 0.0 if two-way STOP controlled;

$I_{b,int}$ = bicycle LOS score for intersection; and

$N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points).

The count of access point approaches used in Equation 17-45 includes both public street approaches and driveways on the right side of the segment in the subject direction of travel.

Step 8: Determine Segment LOS

The bicycle LOS for the segment is determined by using the segment bicycle LOS score from Step 7. This performance measure is compared with the thresholds in Exhibit 17-4 to determine the LOS for the specified direction of travel along the subject segment.

TRANSIT MODE

This subsection describes the methodology for evaluating the performance of an urban street segment in terms of its service to transit passengers.

Urban street segment performance from a transit-passenger perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.*

The methodology is applicable to public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. Procedures for estimating transit vehicle performance on grade-separated or non-public-street rights-of-way, along with procedures for estimating origin–destination service quality, are provided in the *Transit Capacity and Quality of Service Manual* (*3*).

The transit methodology is applied through a series of six steps that culminate in the determination of segment LOS. These steps are illustrated in Exhibit 17-22. Performance measures that are estimated include transit travel speed along the street, transit wait–ride score, and a LOS score reflective of all transit service stopping within or near the segment.

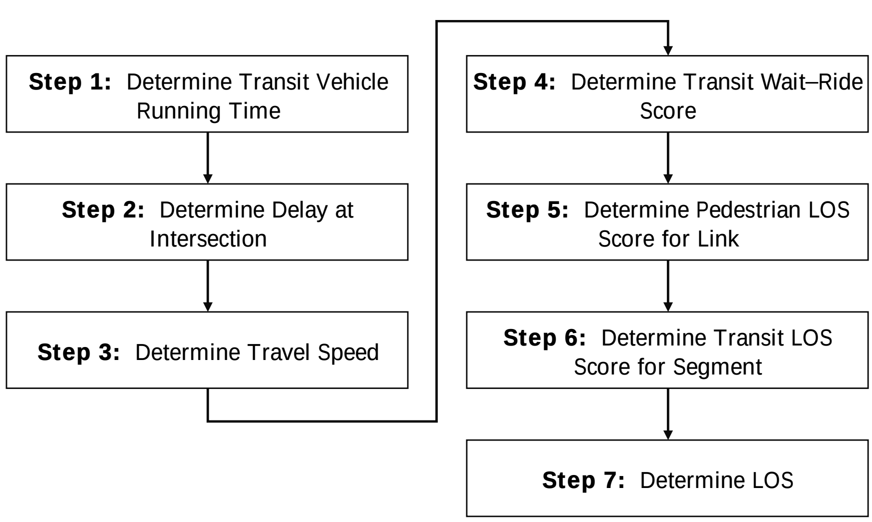


Exhibit 17-22
Transit Methodology for Urban Street Segments

Step 1: Determine Transit Vehicle Running Time

There are two principal components of the transit vehicle's segment running time. One component represents the time required to travel the segment without stopping. (To allow direct comparison with automobile segment speeds, transit vehicles are treated as if they travel the entire segment, even if they join midlink.) The second component is the delay incurred at the transit stops that are provided on the link. The following subparts to this step describe procedures that are used to calculate these components. They culminate with a subpart that describes the calculation of transit vehicle segment running time.

A. Compute Segment Running Speed

Transit vehicle segment running speed represents the speed reached by the vehicle when not influenced by the proximity of a transit stop or traffic control device. This speed can be computed by using Equation 17-46, which is derived from tables given in a Transit Cooperative Research Program report (*8*).

Equation 17-46

$$S_{Rt} = \min\left(S_R, \frac{61}{1 + e^{-1.00 + (1{,}185\, N_{ts} / L)}}\right)$$

where

S_{Rt} = transit vehicle running speed (mi/h),

L = segment length (ft),

N_{ts} = number of transit stops on the segment for the subject route (stops),

S_R = motorized vehicle running speed = (3,600 L)/(5,280 t_R) (mi/h), and

t_R = segment running time (s).

The segment running time is computed by using Equation 17-6 in Step 2 of the automobile methodology.

B. Compute Delay due to a Stop

The delay due to a transit vehicle stop for passenger pickup includes the following components:

- Acceleration–deceleration delay,
- Delay due to serving passengers, and
- Reentry delay.

This procedure is applied once for each stop on the segment. The delay due to each stop is added (in a subsequent step) to compute the total delay due to all stops on the segment.

Acceleration–Deceleration Delay

Acceleration–deceleration delay represents the additional time required to decelerate to stop and then accelerate back to the transit vehicle running speed S_{Rt}. It is computed by using Equation 17-47 and Equation 17-48.

Equation 17-47

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{S_{Rt}}{2}\right)\left(\frac{1}{r_{at}} + \frac{1}{r_{dt}}\right) f_{ad}$$

with

$$f_{ad} = \begin{cases} 1.00 & \text{(stops not on the near side of a boundary intersection)} \\ 0.00 & \text{(near-side stops at all-way and major-street two-way STOP-controlled intersections)} \\ 1-x & \text{(near-side stops at roundabouts)} \\ g/C & \text{(near-side stops at traffic signals)} \end{cases}$$

Equation 17-48

where

d_{ad} = transit vehicle acceleration–deceleration delay due to a transit stop (s),

r_{at} = transit vehicle acceleration rate = 4.0 (ft/s^2),

r_{dt} = transit vehicle deceleration rate = 4.0 (ft/s^2),

f_{ad} = proportion of transit vehicle stop acceleration–deceleration delay not due to traffic control,

x = volume-to-capacity ratio of the link's rightmost lane on a roundabout approach,

g = effective green time (s), and

C = cycle length (s).

Acceleration–deceleration delay represents travel time that is in excess of that required to traverse the equivalent distance at the running speed. It is incurred when the transit vehicle stops solely because of a transit stop. When a transit stop is located on the near side of a boundary intersection, a transit vehicle might need to stop anyway due to the traffic control. In this situation, acceleration–deceleration delay is already included in the through delay estimate (addressed in a subsequent step) and should not be included in d_{ad}. Equation 17-48 is used to determine the proportion of d_{ad} incurred solely because of a transit stop.

If representative acceleration and deceleration rates are known, then they should be used in Equation 17-47. If these rates are unknown, then a rate of 4.0 ft/s^2 may be assumed for both acceleration and deceleration (*8*).

Delay due to Serving Passengers

The delay due to serving passengers is based on the average dwell time, which is an input to this procedure. At signalized intersections, a portion of the dwell time may overlap time the transit vehicle would have spent stopped anyway due to the traffic control. Equation 17-49 is used to compute the delay due to serving passengers.

$$d_{ps} = t_d f_{dt}$$

Equation 17-49

where

d_{ps} = transit vehicle delay due to serving passengers (s),

t_d = average dwell time (s), and

f_{dt} = proportion of dwell time occurring during effective green (= g/C at near-side stops at signalized intersections and 1.00 otherwise, where g and C are as previously defined).

Reentry Delay

The final component of transit vehicle stop delay is the reentry delay d_{re}, which is an input to this procedure. Guidance for estimating reentry delay is provided in the Required Input Data subsection of Section 1, Introduction.

Delay due to a Stop

Delay due to a transit stop is the sum of acceleration–deceleration delay, passenger service time delay, and reentry delay. It is computed by using Equation 17-50.

Equation 17-50

$$d_{ts} = d_{ad} + d_{ps} + d_{re}$$

where d_{ts} = delay due to a transit vehicle stop (s), d_{re} = reentry delay (s), and other variables are as previously defined.

C. Compute Segment Running Time

Equation 17-51 is used to compute transit vehicle running time, which is based on segment running speed and delay due to stops on the segment.

Equation 17-51

$$t_{Rt} = \frac{3{,}600\ L}{5{,}280\ S_{Rt}} + \sum_{i=1}^{N_{ts}} d_{ts,i}$$

where t_{Rt} = segment running time of transit vehicle (s), $d_{ts,i}$ = delay due to a transit vehicle stop for passenger pickup at stop i within the segment (s), and other variables are as previously defined.

If there are no stops on the segment, then the second term of Equation 17-51 equals zero.

Step 2: Determine Delay at Intersection

The through delay incurred at the boundary intersection is determined in this step. This delay is that incurred by the through movement that exits the segment at the downstream boundary intersection. Guidance for determining this delay is provided in Step 5 of the automobile methodology. Equation 17-52 can be used for a planning analysis to estimate the through delay due to a traffic signal (*8*).

Equation 17-52

$$d_t = t_l\ 60 \left(\frac{L}{5{,}280} \right)$$

where

d_t = through delay (s/veh),

t_l = transit vehicle running time loss (min/mi), and

L = segment length (ft).

The running time loss t_l used in Equation 17-52 is obtained from Exhibit 17-23.

Exhibit 17-23
Transit Vehicle Running Time Loss

Area Type	Transit Lane Allocation	Traffic Condition	Running Time Loss by Signal Condition (min/mi): Typical	Signals Set for Transit	Signals More Frequent Than Transit Stops
Central business district	Exclusive	No right turns	1.2	0.6	1.5–2.0
		With right-turn delay	2.0	1.4	2.5–3.0
		Blocked by traffic	2.5–3.0	Not available	3.0–3.5
	Mixed traffic	Any	3.0	Not available	3.5–4.0
Other	Exclusive	Any	0.7 (0.5–1.0)	Not available	Not available
	Mixed traffic	Any	1.0 (0.7–1.5)	Not available	Not available

Source: St. Jacques and Levinson (*8*).

Step 3: Determine Travel Speed

Transit travel speed represents an aggregate measure of speed along the street. It combines the delay incurred at the downstream intersection with the segment running time. As such, it is typically slower than the running speed. The transit travel speed is computed by using Equation 17-53.

Equation 17-53

$$S_{Tt,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rt} + d_t)}$$

where $S_{Tt,seg}$ = travel speed of transit vehicles along the segment (mi/h), t_{Rt} = segment running time of transit vehicle (s), and other variables are as previously defined.

Step 4: Determine Transit Wait–Ride Score

The transit wait–ride score is a performance measure that combines perceived time spent waiting for the transit vehicle and perceived travel time rate. If transit service is not provided for the subject direction of travel, then this score equals 0.0 and the analysis continues with Step 5.

The procedure for calculating the wait–ride score is described in this step. It consists of the separate calculation of the headway factor and the perceived travel time factor. The following subparts describe these two calculations, which culminate in the calculation of the wait–ride score.

A. Compute Headway Factor

The headway factor is the ratio of the estimated patronage at the prevailing average transit headway to the estimated patronage at a base headway of 60 min. The patronage values for the two headways (i.e., the input headway and the base headway of 60 min) are computed from an assumed set of patronage elasticities that relate the percentage change in ridership to the percentage change in headway. The headway factor is computed by using Equation 17-54.

Equation 17-54

$$F_h = 4.00\ e^{-1.434/(v_s + 0.001)}$$

where

F_h = headway factor, and

v_s = transit frequency for the segment (veh/h).

The transit frequency v_s is an input to this procedure. Guidance for estimating this input is provided in the Required Input Data subsection.

B. Compute Perceived Travel Time Factor

Segment performance, as measured by the wait–ride score, is influenced by the travel time rate provided to transit passengers. The perceptibility of this rate is further influenced by the extent to which the transit vehicle is late, crowded, or both and whether the stop provides passenger amenities. In general, travel at a high rate is preferred, but travel at a lower rate may be nearly as acceptable if the transit vehicle is not late, the bus is lightly loaded, and a shelter (with a bench) is provided at the transit stop.

The perceived travel time factor is based on the perceived travel time rate and the expected ridership elasticity with respect to changes in the perceived travel time rate. This factor is computed by using Equation 17-55.

Equation 17-55

$$F_{tt} = \frac{(e-1)\,T_{btt} - (e+1)\,T_{ptt}}{(e-1)\,T_{ptt} - (e+1)\,T_{btt}}$$

with

Equation 17-56

$$T_{ptt} = \left(a_1 \frac{60}{S_{Tt,seg}} \right) + \left(2\,T_{ex} \right) - T_{at}$$

Equation 17-57

$$a_1 = \begin{cases} 1.00 & F_l \le 0.80 \\ 1 + \dfrac{(4)(F_l - 0.80)}{4.2} & 0.80 < F_l \le 1.00 \\ 1 + \dfrac{(4)(F_l - 0.80) + (F_l - 1.00)(6.5 + [(5)(F_l - 1.00)])}{4.2 \times F_l} & F_l > 1.00 \end{cases}$$

Equation 17-58

$$T_{at} = \frac{1.3\,p_{sh} + 0.2\,p_{be}}{L_{pt}}$$

where

F_{tt} = perceived travel time factor;

e = ridership elasticity with respect to changes in the travel time rate = -0.40;

T_{btt} = base travel time rate = 6.0 for the central business district of a metropolitan area with 5 million persons or more, otherwise = 4.0 (min/mi);

T_{ptt} = perceived travel time rate (min/mi);

T_{ex} = excess wait time rate due to late arrivals (min/mi) = t_{ex}/L_{pt};

t_{ex} = excess wait time due to late arrivals (min);

T_{at} = amenity time rate (min/mi);

a_1 = passenger load weighting factor;

$S_{Tt,seg}$ = travel speed of transit vehicles along the segment (mi/h);

F_l = average passenger load factor (passengers/seat);

L_{pt} = average passenger trip length = 3.7 typical (mi);

p_{sh} = proportion of stops on segment with shelters (decimal); and

p_{be} = proportion of stops on segment with benches (decimal).

The perceived travel time rate is estimated according to three components, as shown in Equation 17-56. The first component reflects the average travel speed of the transit service, adjusted for the degree of passenger loading. The second component reflects the average excess wait time for the transit vehicle (i.e., the amount of time spent waiting for a late arrival beyond the scheduled arrival time). The third component reflects the ability of passengers to tolerate longer travel time rates when there are amenities provided at the transit stops.

The first term in Equation 17-56 includes a factor that adjusts the transit vehicle travel time rate by using a passenger load weighting factor. This factor accounts for the decrease in passenger comfort when transit vehicles are crowded. Values of this factor range from 1.00 when the passenger load factor is less than 0.80 passengers/seat to 2.32 when the load factor is 1.6 passengers/seat.

The second term in Equation 17-56 represents the perceived excess wait time rate. It is based on the excess wait time t_{ex} associated with late transit arrivals. The multiplier of 2 in Equation 17-56 is used to amplify the excess wait time rate because passengers perceive excess waiting time to be more onerous than actual travel time.

The excess wait time t_{ex} reflects transit vehicle reliability. It is an input to this procedure. If excess wait time data are not available for a stop, but on-time performance data are available for routes using the stop, then Equation 17-59 may be used to estimate the average excess wait time.

Equation 17-59

$$t_{ex} = [t_{late}(1 - p_{ot})]^2$$

where

t_{ex} = excess wait time due to late arrivals (min),

t_{late} = threshold late time = 5.0 typical (min), and

p_{ot} = proportion of transit vehicles arriving within the threshold late time (default = 0.75) (decimal).

The third term in Equation 17-56 represents the amenity time rate reduction. This rate is computed in Equation 17-58 as the equivalent time value of various

transit stop improvements divided by the average passenger trip length. If multiple transit stops exist on the segment, an average amenity time rate should be used for the segment, based on the average value for all stops in the segment.

The average passenger trip length is used to convert time values for excess wait time and amenities into distance-weighted travel time rates that adjust the perceived in-vehicle travel time rate. The shorter the trip, the greater the influence that late transit vehicles and stop amenities have on the overall perceived speed of the trip.

The average passenger trip length should be representative of transit routes using the subject segment. A value of 3.7 mi is considered to be nationally representative. More accurate local values can be obtained from the National Transit Database (*4*). Specifically, this database provides annual passenger miles and annual unlinked trips in the profile of most transit agencies. The average passenger trip length is computed as the annual passenger miles divided by the annual unlinked trips.

C. Compute Wait–Ride Score

The wait–ride score is computed by using Equation 17-60. A larger score corresponds to better performance.

Equation 17-60

$$s_{w-r} = F_h \; F_{tt}$$

where

s_{w-r} = transit wait–ride score,

F_h = headway factor, and

F_{tt} = perceived travel time factor.

Step 5: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link $I_{p,link}$ is computed by using the pedestrian methodology, as described in a previous subsection.

Step 6: Determine Transit LOS Score for Segment

The transit LOS score for the segment is computed by using Equation 17-61.

Equation 17-61

$$I_{t,seg} = 6.0 - 1.50 \; s_{w-r} + 0.15 \; I_{p,link}$$

where $I_{t,seg}$ is the transit LOS score for the segment and other variables are as defined previously.

Step 7: Determine LOS

The transit LOS is determined by using the transit LOS score from Step 6. This performance measure is compared with the thresholds in Exhibit 17-4 to determine the LOS for the specified direction of travel along the subject street segment.

3. APPLICATIONS

DEFAULT VALUES

Agencies that use the methodologies in this chapter are encouraged to develop a set of local default values based on field measurements on streets in their jurisdiction. Local default values provide the best means of ensuring accuracy in the analysis results. In the absence of local default values, the values identified in this subsection can be used if the analyst believes that they are reasonable for the street segment to which they are applied.

Exhibit 17-5 and Exhibit 17-6 identify the input data variables associated with the automobile, pedestrian, bicycle, and transit methodologies. These variables can be categorized as either (*a*) suitable for specification as a default value or (*b*) required input data. Those variables categorized as “suitable for specification as a default value” have a minor effect on performance estimates and tend to have a relatively narrow range of typical values used in practice. In contrast, required input variables have either a notable effect on performance estimates or a wide range of possible values. Variables suitable for default value specification are discussed in this subsection.

Required input variables typically represent fundamental segment and intersection geometric elements and demand flow rates. Values for these variables should be field measured whenever possible.

If field measurement of the input variables is not possible, then various options exist for determining an appropriate value for a required input variable. As a first choice, input values should be established through the use of local guidelines. If local guidelines do not address the desired variable, then some input values may be determined by considering the typical operation of (or conditions at) similar segments and intersections in the jurisdiction. As a last option, various authoritative national guideline documents are available and should be used to make informed decisions about design options and volume estimates. The use of simple rules of thumb or “ballpark” estimates for required input values is discouraged because this use is likely to lead to a significant cumulative error in the performance estimates.

Automobile Mode

The required input variables for the automobile methodology are identified in the following list. These variables represent the minimum basic input data that the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 17-5.

- Demand flow rate (at boundary intersection),
- Capacity (at boundary intersection),
- Number of lanes (at boundary intersection),
- Upstream intersection width (at boundary intersection),
- Turn bay length (at boundary intersection),
- Number of through lanes,

- Segment length,
- Restrictive median length (if present),
- Speed limit,
- Through control delay (at boundary intersection),
- Through stopped vehicles (at boundary intersection), and
- Second- and third-term back of queue (at boundary intersection).

Several authoritative reference documents (*9–11*) provide useful guidelines for selecting the type of signal control at the boundary intersection and determining the appropriate traffic control for the segment.

Exhibit 17-24 lists default values for the automobile methodology. Some of the values listed may also be useful for the pedestrian, bicycle, or transit methodologies. The last column of this exhibit indicates "see discussion" for one variable. In this situation, the default value is described in the discussion provided in this subsection.

Exhibit 17-24
Default Values: Automobile Mode

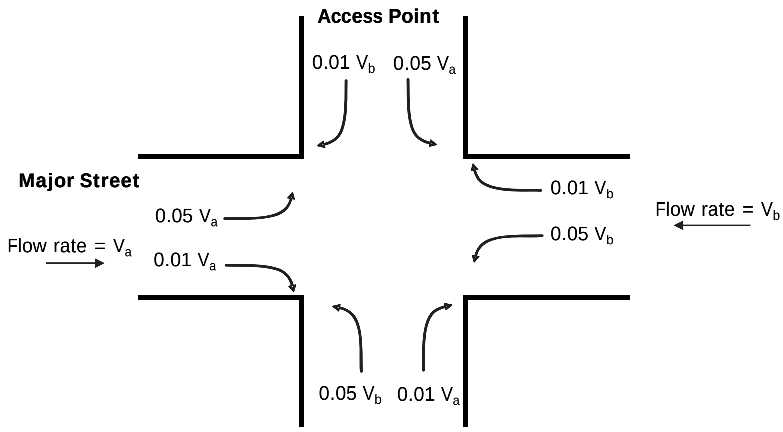
Data Category	Input Data Element	Default Values
Traffic characteristics	Access point flow rate	See discussion
	Midsegment flow rate	Estimate by using demand flow rate at the downstream boundary intersection approach
Geometric design	Number of lanes at access points	Segment Approach If median is present, one left-turn lane/approach. If no median is present, no left-turn lanes. No right-turn lanes. Through lanes are the same as N_{th}. Access Point Approach One left-turn lane and one right-turn lane.
	Turn bay length at access points	40% of the access point spacing, where spacing equals 2 (5,280) / D_{ap} in feet. Computed bay length should not exceed 300 ft or be less than 50 ft.
	Proportion of segment with curb	1.0 (curb present on both sides of segment)
	Number of access point approaches	Estimated for each segment side by multiplying default access point density by 1/2 segment length (i.e., $N_{ap,s}$ = 0.5 D_{ap} L / 5,280) Urban arterial D_{ap} = 34 points/mi Suburban arterial D_{ap} = 21 points/mi Urban collector D_{ap} = 61 points/mi Suburban collector D_{ap} = 48 points/mi
Other	Analysis period duration	0.25 h
Performance measures	Midsegment delay	0.0 s/veh
	Midsegment stops	0.0 stops/veh

Note: D_a = access point density on segment (points/mi); $N_{ap,s}$ =number of access point approaches on the right side in the subject direction of travel (points); L = segment length (ft); and N_{th} = number of through lanes on the segment in the subject direction of travel (ln).

The default access point flow rate can be estimated from the midsegment flow rate by using default turn proportions. These proportions are shown in Exhibit 17-25 for a typical access point intersection on an arterial street. The proportion of 0.05 for the left-turn movements can be reduced to 0.01 for a typical access point on a collector street. These proportions are appropriate for

segments with an access point density consistent with the default densities in Exhibit 17-24 and are applicable to access points serving any public-oriented land use (this excludes single-family residential land use and undeveloped property).

Exhibit 17-25
Default Turn Proportions for Access Point Intersections



If one of the movements shown in Exhibit 17-25 does not exist at a particular access point intersection, then its volume is not computed (its omission has no effect on the proportion used for the other movement flow rates). The flow rate for the crossing movements at an access point intersection is not needed for the automobile methodology. The left-turn proportions shown are larger than the right-turn proportions because right-turn opportunities are typically more frequent than left-turn opportunities along an arterial street.

Nonautomobile Modes

The required input variables for the pedestrian, bicycle, and transit methodologies are identified in the list below. These variables represent the minimum basic input data that the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 17-6.

<u>Pedestrian Methodology</u>

- Midsegment flow rate
- Pedestrian flow rate
- Downstream intersection width (at boundary intersection)
- Segment length
- Number of through lanes
- Median type and curb presence
- Spacing of objects in buffer
- Legality of midsegment pedestrian crossing
- Proportion of segment adjacent to window display
- Proportion of segment adjacent to building face
- Proportion of segment adjacent to low wall or fence
- Motorized vehicle running speed
- Pedestrian delay

- Pedestrian LOS score for intersection

Bicycle Methodology

- Midsegment flow rate
- Segment length
- Number of through lanes
- Median type and curb presence
- Motorized vehicle running speed
- Bicycle delay (at boundary intersection)
- Bicycle LOS score for intersection

Transit Methodology

- Excess wait time (or on-time performance)
- Transit frequency
- Segment length
- Area type
- Transit stop location
- Transit stop position
- Proportion of stops with shelters
- Proportion of stops with benches
- Motorized vehicle running speed
- Pedestrian LOS score for link
- Through control delay (at boundary intersection)
- Reentry delay
- Effective green-to-cycle-length ratio (at boundary intersection)
- Volume-to-capacity ratio (at roundabout boundary intersection)

Exhibit 17-26 lists the default values for the pedestrian, bicycle, and transit methodologies (*2*, *12*).

TYPES OF ANALYSIS

The automobile, pedestrian, bicycle, and transit methodologies described in this chapter can each be used in three types (or levels) of analysis. These analysis levels are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis level are described in the subsequent parts of this subsection.

Operational Analysis

Each of the methodologies is most easily applied at an operational level of analysis. At this level, all traffic, geometric, and signalization conditions are specified as input variables by the analyst. These input variables are used in the methodology to compute various performance measures.

Exhibit 17-26
Default Values: Nonautomobile Modes

Data Category	Input Data Element	Default Value
Traffic characteristics	Dwell time	Downtown stop, transit center, major on-line transfer point, major park-and-ride stop: 60 s Major outlying stop: 30 s Typical outlying stop: 15 s
	Passenger trip length	3.7 mi
	Passenger load factor	0.80 passengers/seat
	Percent heavy vehicles	3%
	Proportion of on-street parking occupied	0.50 (if parking lane present)
Geometric design	Width of outside through lane	12 ft
	Width of bicycle lane	5.0 ft (if provided)
	Width of paved outside shoulder	No parking lane: 1.5 ft (curb and gutter width) Parking lane present: 8.0 ft
	Number of access point approaches	Estimated for each segment side by multiplying default access point density by segment length (i.e., $N_{ap,s} = 0.5\ D_{ap}\ L$ / 5,280) Urban arterial D_{ap} = 34 points/mi Suburban arterial D_{ap} = 21 points/mi Urban collector D_{ap} = 61 points/mi Suburban collector D_{ap} = 48 points/mi
	Total walkway width	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
	Effective width of fixed objects	Business or office land use: 2.0 ft inside, 2.0 ft outside Residential or industrial land use: 0.0 ft inside, 0.0 ft outside
	Buffer width	Business or office land use: 0.0 ft Residential or industrial land use: 6.0 ft
Other	Pavement condition rating	3.5
	Distance to nearest signal-controlled crossing	One-third the distance between signal-controlled crossings that bracket the segment
Performance measures	Delay at midsegment signalized crosswalk	20 s/p

Note: D_a = access point density on segment (points/mi); $N_{ap,s}$ =number of access point approaches on the right side in the subject direction of travel (points); L = segment length (ft); and N_{th} = number of through lanes on the segment in the subject direction of travel (ln).

Design Analysis

The nature of the design analysis varies depending on whether the boundary intersections are unsignalized or signalized. When the segment has unsignalized boundary intersections, the analyst specifies traffic conditions and target levels for a specified set of performance measures. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

When the segment has signalized boundary intersections, the design level of analysis has two variations. Both variations require the specification of traffic conditions and target levels for a specified set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

The second variation of the design level requires the additional specification of the geometric conditions. The methodology is then applied by using an iterative approach in which alternative signalization conditions are evaluated.

The objective of the design analysis is to identify the alternatives that operate at the target level of the specified performance measures (or provide a better level of performance). The analyst may then recommend the "best" design alternative after consideration of the full range of factors.

Planning and Preliminary Engineering Analysis

The planning and preliminary engineering level of analysis is intended to provide an estimate of the LOS for either a proposed segment or an existing segment in a future year. This level of analysis may also be used to size the overall geometrics of a proposed segment.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose were described previously in this section.

The requirement for a complete description of the signal timing plan can be a burden for some planning analyses involving signalized intersections, especially when the signal control is pretimed or coordinated–actuated. The intersection Quick Estimation Method described in Chapter 31, Signalized Intersections: Supplemental, can be used to estimate a reasonable timing plan, in conjunction with the aforementioned default values.

For some planning and preliminary engineering analyses, the segment Quick Estimation Method described in Chapter 30, Urban Street Segments: Supplemental, may provide a better balance between accuracy and analysis effort in the evaluation of vehicle LOS.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for the application of alternative tools to the analysis of urban street segments. The tools are described as either simulation or deterministic, in reference to their traffic modeling approach. Additional information on this topic is provided in Volume 4. The focus of this section is the application of alternative tools to evaluate automobile operation.

Strengths of the Automobile Methodology

The automobile methodology described in Section 2 models the driver–vehicle–road system with reasonable accuracy for most applications. It accounts for midsegment speed variations due to traffic and geometric conditions. Alternative tools offer a more detailed treatment of the arrival and departure of vehicles as well as the interaction between the vehicle, the roadway, and the

control system. As such, some tools can model the driver–vehicle–road system more accurately for some applications.

The automobile methodology offers several advantages over alternative analysis tools. One advantage is that it has an empirically calibrated procedure for estimating saturation flow rate. Alternative tools often require saturation flow rate as an input variable. A second advantage is that it produces a direct estimate of capacity and volume-to-capacity ratio. These measures are not directly available from simulation tools. A third advantage is that it produces an expected value for each of a wide variety of data outputs in a single application. Many alternative tools operate as a "black box," providing little detail describing the intermediate calculations. Moreover, simulation tools require multiple runs and manual calculations to obtain expected values for the output data.

Identified Limitations of the Automobile Methodology

The limitations of the automobile methodology are identified in Section 1. If any of these limitations apply to a particular situation, then alternative tools may produce more credible performance estimates. Limitations involving consideration of the impact of progression on performance are a special case that is discussed in more detail in Chapter 16, Urban Street Facilities.

Features and Performance Measures Available from Alternative Tools

Both deterministic tools and simulation tools are in common use as alternatives to the procedures offered in this chapter. Deterministic tools are used to a greater extent for the analysis of urban street segments than for most of the other transportation elements represented in this manual. The main reasons for their popularity are found in the user interface, optimization options, and output presentation features. Some also offer additional performance measures such as fuel consumption, air quality, and operating cost.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS assessment for the automobile mode on urban street segments is based on the average travel speed over the segment. The average travel speed is computed by dividing the segment length by the total time required to travel the segment, taking into account all intersection and nonintersection delays.

Alternative tools generally define the travel speed in the same way that it is defined in this chapter. However, these tools may not compute delay and running speed by using the procedures presented in Section 2. Therefore, some care must be taken when using speed and delay estimates from other tools. Issues related to speed and delay comparison among different tools are discussed in more detail in Chapter 7. In general, the travel speed from an alternative tool should not be used for LOS assessment unless the tool is confirmed to apply the procedures described in Section 2.

Conceptual Differences That Preclude Direct Comparison of Results

Alternative deterministic tools apply traffic models that are conceptually similar to those described in this chapter. While their computational details will

usually produce different numerical results, there are few major conceptual differences that would preclude comparison in terms of relative magnitude.

Simulation tools, on the other hand, are based on entirely different modeling concepts. A general discussion of the conceptual differences is presented in Chapters 6 and 7. Some specific examples for signalized intersections, which also apply to urban street segments, are presented in Chapter 18.

One phenomenon that makes comparison difficult is the propagation of platoons along a segment. Deterministic tools, including the model presented in this chapter, apply equations that spread out a platoon as it progresses downstream. Simulation tools create platoon dispersion implicitly from a distribution of desired speeds among drivers. Both approaches will produce platoon dispersion, but the amount of dispersion will differ among tools.

Simulation tools may also exhibit platoon compression because of the effect of slower-moving vehicles that cause platoons to regenerate. For this and other reasons, it is difficult to achieve comparability of platoon representation along a segment between these tools and the automobile methodology.

Adjustment of Alternative Tool Parameters

For applications in which either an alternative tool or the automobile methodology can be used, some adjustment will generally be required for the alternative tool if some consistency with the automobile methodology is desired. For example, the parameters that determine the capacity of a signalized approach (e.g., saturation flow rate and start-up lost time) should be adjusted to ensure that the lane group (or approach) capacities match those estimated by the automobile methodology.

It might also be necessary to adjust the parameters that affect the travel time along the segment to produce comparable results. The automobile methodology is based on a free-flow speed that is computed as a function of demand flow rate, median type, access point density, and speed limit. Most alternative tools typically require a user-specified free-flow speed, which could be obtained from the automobile methodology to maintain comparability. It may be more difficult to adjust the platoon modeling parameters. So, if comparability is desired in representing the platoon effect, it is preferable to adjust the free-flow speed specified for simulation such that the actual travel speeds are similar to those obtained from the automobile methodology.

Step-by-Step Recommendations for Applying Alternative Tools

A set of step-by-step recommendations for signalized intersection evaluation with alternative tools is presented in Chapter 18. The recommendations in that chapter also apply to the evaluation of urban street segments.

Sample Calculations Illustrating Alternative Tool Applications

The most useful examples of the application of alternative tools involve multiple segment facilities. Chapter 29, Urban Street Facilities: Supplemental, includes a set of examples to illustrate the use of alternative tools to address the stated limitations of this chapter and Chapter 16, Urban Street Facilities.

Specifically, these examples illustrate (*a*) the application of deterministic tools to optimize signal timing, (*b*) the effect of using a roundabout as a segment boundary, (*c*) the effect of midsegment parking maneuvers on facility operation, and (*d*) the use of simulated vehicle trajectories to evaluate the proportion of time that the back of the queue on the minor-street approach to a two-way STOP-controlled intersection exceeds a specified distance from the stop line.

Chapter 31, Signalized Intersections: Supplemental, includes example problems that address left-turn storage bay overflow, right-turn-on-red operation, short through lanes, and closely spaced intersections.

4. EXAMPLE PROBLEMS

This part of the chapter describes the application of each of the automobile, pedestrian, bicycle, and transit methodologies through the use of example problems. Exhibit 17-27 provides an overview of these problems. The focus of the examples is on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operational analysis level in terms of the calculations, except that default values are used when field-measured values are not available.

Exhibit 17-27
Example Problems

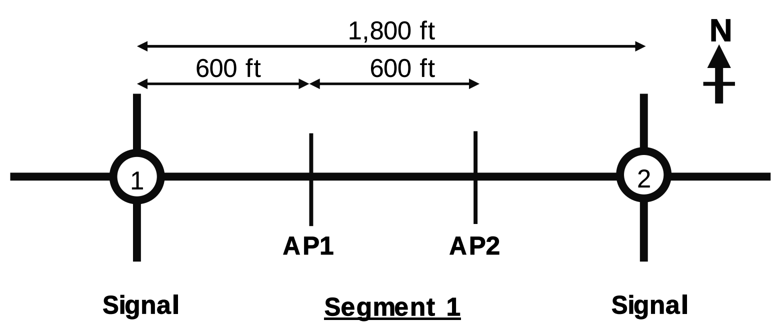
Problem Number	Description	Analysis Level
1	Automobile LOS	Operational
2	Pedestrian LOS	Operational
3	Bicycle LOS	Operational
4	Transit LOS	Operational

EXAMPLE PROBLEM 1: AUTOMOBILE LOS

The Urban Street Segment

The total length of an undivided urban street segment is 1,800 ft. It is shown in Exhibit 17-28. Both of the boundary intersections are signalized. The street has a four-lane cross section with two lanes in each direction. There are left-turn bays on the subject segment at each signalized intersection.

Exhibit 17-28
Example Problem 1: Urban Street Segment Schematic



The segment has two access point intersections, shown in the exhibit as AP1 and AP2. Each intersection has two STOP-controlled side-street approaches. The segment has some additional driveways on each side of the street; however, their turn movement volumes are too low during the analysis period for them to be considered "active." So, the few vehicles that do turn at these locations during the analysis period have been added to the appropriate flow rates at the two access point intersections.

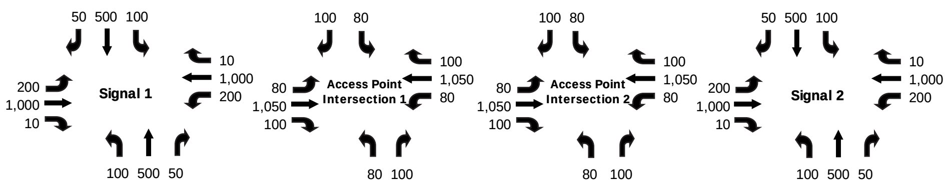
The Question

What are the travel speed, spatial stop rate, and LOS during the analysis period for the segment through movement in both directions of travel?

The Facts

The segment's traffic counts are listed in Exhibit 17-29. The counts were taken during the 15-min analysis period of interest. However, they have been converted to hourly flow rates. It is noted that the volumes leaving the signalized intersections do not add to equal the volume arriving at the downstream access point intersection.

Exhibit 17-29
Example Problem 1: Intersection Turn Movement Counts



The signalization conditions are shown in Exhibit 17-30. The conditions shown are identified as belonging to Signalized Intersection 1; however, they are the same for Signalized Intersection 2. The signals operate with coordinated–actuated control. The left-turn movements on the northbound and southbound approaches operate under protected–permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the major street operate as protected-only in a lead–lead sequence.

Exhibit 17-30
Example Problem 1: Signal Conditions for Intersection 1

Signalized Intersection 1								
General Information								
Cross Street:	First Avenue			**Analysis Period:**	7:15 am to 7:30 am			
Phase Sequence and Left-Turn Mode								
Major street sequence (movement numbers shown)	5 & 1 left leading			Cross street sequence (movement numbers shown)	3 & 7 left leading			
Major street left-turn mode (movement numbers shown)	5/1 Protected-Only			Cross street left-turn mode (movement numbers shown)	3/7 Protected+Permitted			
Phase Settings								
Approach	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Phase number	5	2	1	6	3	8	7	4
Movement	L	T+R	L	T+R	L	T+R	L	T+R
Lead/lag left-turn phase	Lead	--	Lead	--	Lead	--	Lead	--
Left-turn mode	Prot.	--	Prot.	--	Pr/Pm	--	Pr/Pm	--
Passage time, s	2.0	--	2.0	--	2.0	2.0	2.0	2.0
Minimum green, s	5	--	5	--	5	5	5	5
Yellow + red clear, s	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Phase split, s	20	35	20	35	20	25	20	25
Recall	No	--	No	--	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes
Ref. Phase: 2	Offset, s: 0		Offset Ref.: End of Green		Force Mode: Fixed			
	Cycle, s: 100							
Enable Simultaneous Gap-Out? Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑				Enable Dallas Left-Turn Phasing? Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐				

Exhibit 17-30 indicates that the passage time for each phase is 2.0 s. The minimum green setting is 5 s for each phase. The offset to Phase 2 (the reference phase) end-of-green interval is 0.0 s. A fixed-force mode is used to ensure that coordination is maintained. The cycle length is 100 s.

Geometric conditions and traffic characteristics for Signalized Intersection 1 are shown in Exhibit 17-31. They are the same for Signalized Intersection 2. The

movement numbers follow the numbering convention shown in Exhibit 18-2 of Chapter 18.

Exhibit 17-31
Example Problem 1: Geometric Conditions and Traffic Characteristics for Signalized Intersection 1

Signalized Intersection 1												
Signalized Intersection Input Data *(In each column, enter the volume and lanes data. For all other blue cells, enter values only if there is one or more lanes.)*												
Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Volume, veh/h	200	1,000	10	200	1,000	10	100	500	50	100	500	50
Lanes	1	2	1	1	2	1	1	2	0	1	2	0
Turn bay length, ft	200		200	200		200	200			200		
Sat. flow rate, veh/h/ln	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800		1,800	1,800	
Platoon ratio	1.000	1.333	1.000	1.000	1.333	1.000	1.000	1.000		1.000	1.000	
Initial queue, veh	0	0	0	0	0	0	0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Stop line det. length, ft	40			40			40	40		40	40	
Max. allow. hdwy, s/ve	3.9			3.9			3.9	2.9		3.9	2.9	

All signalized intersection approaches have a 200-ft left-turn bay and two through lanes. The east–west approaches have a 200-ft right-turn lane. The north–south approaches have a shared through and right-turn lane. The saturation flow rate is determined by using the procedure described in Chapter 18.

The platoon ratio is entered for all movements associated with an external approach to the segment. The eastbound through movement at Signalized Intersection 1 is coordinated with the upstream intersection such that favorable progression occurs, as described by a platoon ratio of 1.33. The westbound through movement at Signalized Intersection 2 is also coordinated with its upstream intersection, and arrivals are described by a platoon ratio of 1.33. Arrivals to all other movements are characterized as "random" and are described with a platoon ratio of 1.00. The movements for the westbound approach at Signalized Intersection 1 (and eastbound approach at Signalized Intersection 2) are internal movements, so the input platoon ratios shown will only be used for the first iteration of calculations. More accurate values are computed during subsequent iterations by using a procedure provided in the methodology.

The speed limit on the segment and on the cross-street approaches is 35 mi/h. A 40-ft detection zone is located just upstream of the stop line in each traffic lane at the two signalized intersections.

The geometric conditions that describe the segment are shown in Exhibit 17-32. These data are used to compute the free-flow speed for the segment.

The traffic and lane assignment data for the two access point intersections are shown in Exhibit 17-33. The movement numbers follow the numbering convention shown in Exhibit 19-3 of Chapter 19, Two-Way STOP-Controlled Intersections. There are no turn bays on the segment at the two access point intersections.

Exhibit 17-32
Example Problem 1: Segment Data

Segment 1		
Free-Flow Speed Computation		
Input Data		
	EB	**WB**
Basic Segment Data		
Number of through lanes that extend the length of the segment:	2	2
Speed limit, mph	35	35
Segment Length Data		
Length of segment (measured stopline to stopline), ft	1,800	1,800
Width of upstream signalized intersection, ft	50	50
Adjusted segment length, ft	1,750	1,750
Length of segment with a restrictive median (e.g, raised-curb), ft	0	0
Length of segment with a non-restrictive median (e.g, two-way left-turn lane), ft	0	0
Length of segment with no median, ft	1,750	1,750
Percentage of segment length with restrictive median, %	0	0
Access Data		
Percentage of street with curb on right-hand side (in direction of travel), %	70	70
Number of access points on right-hand side of street (in direction of travel)	4	4
Access point density, access points/mi	24	24

Exhibit 17-33
Example Problem 1: Access Point Data

Access Point Input Data													
Access	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Point	Movement	L	T	R	L	T	R	L	T	R	L	T	R
Location,ft	Movement number	1	2	3	4	5	6	7	8	9	10	11	12
600	Volume, veh/h	80	1,050	100	80	1,050	100	80	0	100	80	0	100
West end	Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1200	Volume, veh/h	80	1,050	100	80	1,050	100	80	0	100	80	0	100
	Lanes	0	2	0	0	2	0	1	0	1	1	0	1

Outline of Solution

Movement-Based Data

Exhibit 17-34 provides a summary of the analysis of the individual traffic movements at Signalized Intersection 1.

Exhibit 17-34
Example Problem 1: Movement-Based Output Data

INTERSECTION 1	**EB**	**EB**	**EB**	**WB**	**WB**	**WB**	**NB**	**NB**	**NB**	**SB**	**SB**	**SB**
	L	**T**	**R**	**L**	**T**	**R**	**L**	**T**	**R**	**L**	**T**	**R**
Movement:	**5**	**2**	**12**	**1**	**6**	**16**	**3**	**8**	**18**	**7**	**4**	**14**
Volume, veh/h	200	1,000	10	194	968	10	100	500	50	100	500	50
Lanes	1	2	1	1	2	1	1	2	0	1	2	0
Bay Length, ft	200	0	200	200	0	200	200	0	0	200	0	0
Saturation Flow Rate, veh/h/ln	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800
Platoon Ratio	1.00	1.33	1.00	1.00	1.33	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Speed Limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Detector Length, ft	40	0	0	40	0	0	40	40	40	40	40	40
Capacity, veh/h	234	1,703	724	230	1,695	720	213	609	61	213	609	61
Discharge Volume, veh/h	0	1,000	0	0	0	0	0	0	50	100	0	0
Proportion Arriving On Green	0.137	0.630	0.473	0.046	0.456	0.460	0.063	0.189	0.189	0.063	0.189	0.189
Approach Volume, veh/h		1,210			1,172			650			650	
Approach Delay, s/veh		19.4			25.5			39.4			39.4	
Approach Stop Rate, stops/veh		0.471			0.668			0.850			0.850	

With one exception, the first eight rows of data in Exhibit 17-34 are an "echo" of the input data. The remaining rows list variables that are computed by using these input data. The volumes shown in Exhibit 17-34 for the eastbound (EB), northbound (NB), and southbound (SB) movements are identical to the input volumes. The westbound (WB) volumes were reduced from the input volumes during Step 1: Determine Traffic Demand Adjustments. This reduction occurred because the westbound volume input for this intersection exceeded the volume departing the upstream access point intersection (i.e., AP1).

Capacity for a movement is computed by using the movement volume proportion in each approach lane group, lane group saturation flow rate, and corresponding phase duration. This variable represents the capacity of the movement, regardless of whether it is served in an exclusive lane or a shared lane. If the movement is served in a shared lane, then the movement capacity represents the portion of the lane group capacity available to the movement, as distributed in proportion to the flow rate of the movements served by the associated lane group.

Discharge volume is computed for those movements that enter a segment during Step 1: Determine Traffic Demand Adjustments. At Signalized Intersection 1, the movements entering the segment are the eastbound through movement, northbound right-turn movement, and southbound left-turn movement. A value of 0.0 veh/h is shown for all other movements and indicates that they are not relevant to this calculation. If volume exceeds capacity for any given movement, then the discharge volume is set equal to the capacity. Otherwise, the discharge volume is equal to the movement volume.

The proportion arriving during green *P* is computed for internal movements during Step 3: Determine the Proportion Arriving During Green. In contrast, it is computed from the input platoon ratio for external movements.

The last three rows in Exhibit 17-34 represent summary statistics for the approach. The approach volume represents the sum of the three movement volumes. Approach delay and approach stop rate are computed as volume-weighted averages for the lane groups served on an intersection approach.

Timer-Based Phase Data

Exhibit 17-35 provides a summary of the output data for Signalized Intersection 1 using a signal controller perspective. The controller has eight timing functions (or timers), with Timers 1 to 4 representing Ring 1 and Timers 5 to 8 representing Ring 2. The ring structure and phase assignments are described in Chapter 18. Timers 1, 2, 5, and 6 are used to control the east–west traffic movements on the segment. Timers 3, 4, 7, and 8 are used to control the north–south movements that cross the segment.

Exhibit 17-35
Example Problem 1: Timer-Based Phase Output Data

Timer Data

Timer:	**1 WB L**	**2 EB T.R**	**3 NB L**	**4 SB T.T+R**	**5 EB L**	**6 WB T.R**	**7 SB L**	**8 NB T.T+R**
Assigned Phase	1	2	3	4	5	6	7	8
Case No	2	3	1	4	2	3	1	4
Phase Duration (G+Y+Rc), s	16.48	51.29	9.32	22.90	16.69	51.09	9.32	22.90
Change Period (Y+Rc), s	3.00	4.00	3.00	4.00	3.00	4.00	3.00	4.00
Phase Start Time, s	36.22	52.70	4.00	13.32	36.22	52.92	4.00	13.32
Phase End Time, s	52.70	4.00	13.32	36.22	52.92	4.00	13.32	36.22
Max. Allowable Headway (MAH), s	3.13	0.00	3.13	3.06	3.13	0.00	3.13	3.06
Equivalent Maximum Green (Gmax), s	29.78	0.00	17.00	31.68	29.78	0.00	17.00	31.68
Max. Queue Clearance Time (g_c+l1),	13.238	0.000	6.644	16.955	13.432	0.000	6.644	16.955
Green Extension Time (g_e), s	0.302	0.000	0.098	1.946	0.313	0.000	0.098	1.946
Probability of Phase Call (p_c)	0.995	0.000	0.938	1.000	0.996	0.000	0.938	1.000
Probability of Max Out (p_x)	0.000	0.000	0.000	0.023	0.000	0.000	0.000	0.023

Cycle Length, s: 100

The timing function construct is essential to the modeling of a ring-based signal controller. *Timers* always occur in the same numeric sequence (i.e., 1 then 2 then 3 then 4 in Ring 1; 5 then 6 then 7 then 8 in Ring 2). The practice of associating movements with phases (e.g., the major-street through movement to Phase 2), coupled with the occasional need for lagging left-turn phases and split phasing, creates the situation in which *phases* do not always time in sequence. For example, with a lagging left-turn phase sequence, major-street through Phase 2 times first and then major-street left-turn Phase 1 times second.

The modern controller accommodates the assignment of phases to timing functions by allowing the ring structure to be redefined manually or by time-of-

day settings. Specification of this structure is automated in the computational engine by the assignment of phases to timers.

The methodology is based on modeling *timers*, not by directly modeling movements or phases. The methodology converts movement and phase input data into timer input data. It then models controller response to these inputs and computes timer duration and related performance measures.

The two signalized intersections in this example problem have lead–lead left-turn sequences. Hence, the timer number is equal to the phase number (e.g., the westbound movement is associated with Phase 1, which is assigned to Timer 1).

The case number shown in Exhibit 17-35 is used as a single variable descriptor of each possible combination of left-turn mode and lane group type (i.e., shared or exclusive). An understanding of this variable is not needed to interpret the output data.

The phase duration shown in Exhibit 17-35 represents the estimated average phase duration during the analysis period. It represents the sum of the green, yellow change, and red clearance intervals. For Timer 2 (i.e., Phase 2), the average green interval duration can be computed as 47.29 s (= 51.29 – 4.00).

The phase start time represents the time the timer (and phase) starts, relative to system time 0.0. For Phase 2, the start time is 52.70 s. The end of the green interval associated with this phase is 100.0 s (= 52.70 + 47.29). This time is equal to the cycle length, so the end of green actually occurs at 0.0 s. This result is expected because Phase 2 is the coordinated phase and the offset to the end of Phase 2 (relative to system time 0.0) was input as 0.0 s.

The phase end time represents the time the timer (and phase) ends relative to system time 0.0. For Phase 2, the end of the green interval occurs at 0.0 s and the end of the phase occurs 4.0 s later (i.e., the change period duration).

The remaining variables in Exhibit 17-35 apply to the noncoordinated phases (i.e., the actuated phases). These variables describe the phase timing and operation. They are described in more detail in Chapter 18.

Timer-Based Movement Data

Exhibit 17-36 summarizes the output for Signalized Intersection 1 as it relates to the movements assigned to each timer. Separate sections of output are shown in the exhibit for the left-turn, through, and right-turn movements. The assigned movement row identifies the movement (previously identified in Exhibit 17-34) assigned to each timer.

The saturation flow rate shown in Exhibit 17-36 represents the saturation flow rate for the movement. The procedure for calculating these rates is described in Chapter 18. In general, the rate for a movement is the same as for a lane group when the lane group serves one movement. The rate is split between the movements when the lane group is shared by two or more movements.

Exhibit 17-36
Example Problem 1: Timer-Based Movement Output Data

Timer Data								
Timer:	**1 WB L**	**2 EB T.R**	**3 NB L**	**4 SB T.T+R**	**5 EB L**	**6 WB T.R**	**7 SB L**	**8 NB T.T+R**
Left-Turn Movement Data								
Assigned Movement	1		3		5		7	
Mvmt. Sat Flow, veh/h	1,710.00		1,710.00		1,710.00		1,710.00	
Through Movement Data								
Assigned Movement		2		4		6		8
Mvmt. Sat Flow, veh/h		3,600.00		3,222.18		3,600.00		3,222.18
Right-Turn Movement Data								
Assigned Movement		12		14		16		18
Mvmt. Sat Flow, veh/h		1,530.00		321.15		1,530.00		321.15

Timer-Based Lane Group Data

The methodology described in Chapter 18 computes a variety of output data that describe the operation of each intersection lane group. The example problem in Chapter 18 illustrates these data and discusses their interpretation. The output data for the individual lane groups are not repeated in this chapter. Instead, the focus of the remaining discussion is on the access point output and the performance measures computed for the two segment through movements.

Access Point Data

Exhibit 17-37 illustrates the output statistics for the two access point intersections located on the segment. The first six rows listed in the exhibit correspond to Access Point Intersection 1 (AP1), and the second six rows correspond to Access Point Intersection 2 (AP2). Additional sets of six rows would be provided in this table if additional access point intersections were evaluated.

Exhibit 17-37
Example Problem 1: Movement-Based Access Point Output Data

Access Point Data Segment 1 **Movement:**	**EB L 1**	**EB T 2**	**EB R 3**	**WB L 4**	**WB T 5**	**WB R 6**	**NB L 7**	**NB T 8**	**NB R 9**	**SB L 10**	**SB T 11**	**SB R 12**
Access Point Intersection No. 1												
1: Volume, veh/h	74.80	981.71	93.50	75.56	991.70	94.45	80.00	0.00	100.00	80.00	0.00	100.00
1: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1: Proportion time blocked	0.170			0.170			0.260	0.260	0.170	0.260	0.260	0.170
1: Delay to through vehicles, s/veh		0.163			0.164							
1: Prob. inside lane blocked by left		0.101			0.101							
1: Dist. from West/South signal, ft	600											
Access Point Intersection No. 2												
2: Volume, veh/h	75.56	991.70	94.45	74.80	981.71	93.50	80.00	0.00	100.00	80.00	0.00	100.00
2: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
2: Proportion time blocked	0.170			0.170			0.260	0.260	0.170	0.260	0.260	0.170
2: Delay to through vehicles, s/veh		0.164			0.163							
2: Prob. inside lane blocked by left		0.101			0.101							
2: Dist. from West/South signal, ft	1,200											

The eastbound and westbound volumes listed in Exhibit 17-37 are not equal to the input volumes. These volumes were adjusted during Step 1: Determine Traffic Demand Adjustments such that they equal the volume discharging from the upstream intersection. This routine achieves balance between all junction pairs (e.g., between Signalized Intersection 1 and Access Point Intersection 1, between Access Point Intersection 1 and Access Point Intersection 2, and so forth).

The proportion of time blocked is computed during Step 3: Determine the Proportion Arriving During Green. It represents the proportion of time during the cycle that the associated access point movement is blocked by the presence of a platoon passing through the intersection. For major-street left turns, the platoon of concern approaches from the opposing direction. For the minor-street left turn, platoons can approach from either direction and can combine to block this left turn for extended time periods. This trend can be seen by comparing the

proportion of time blocked for the eastbound (major-street) left turn (i.e., 0.17) with that for the northbound (minor-street) left turn (i.e., 0.26) at Access Point Intersection 1.

The delay to through vehicles is computed during Step 2: Determine Running Time. It represents the sum of the delay due to vehicles turning left from the major street and the delay due to vehicles turning right from the major street. This delay tends to be small compared with typical signalized intersection delay values. But it can influence travel speed if there are several high-volume access points on a street and only one or two through lanes in each direction of travel.

The probability of the inside through lane being blocked is also computed during Step 2: Determine Running Time as part of the delay-to-through-vehicles procedure. This variable indicates the probability that the left-turn bay at an access point will overflow into the inside through lane on the street segment. Hence, it indicates the potential for a through vehicle to be delayed by a left-turn maneuver. The segment being evaluated has an undivided cross section, and no left-turn bays are provided at the access point intersections. In this situation, the probability of overflow is 0.10, indicating that the inside lane is blocked about 10% of the time.

Results

Exhibit 17-38 summarizes the performance measures for the segment. Also shown are the results from the spillback check conducted during Step 1: Determine Traffic Demand Adjustments. The movements indicated in the column heading represent the movements exiting the segment at a boundary intersection. Thus, the westbound movements on Segment 1 are those that occur at Signalized Intersection 1. Similarly, the eastbound movements on Segment 1 are those that occur at Signalized Intersection 2.

Exhibit 17-38
Example Problem 1: Performance Measure Summary

Segment Summary Seg.No.	Movement:	EB L 5	EB T 2	EB R 12	WB L 1	WB T 6	WB R 16
1	Bay/Lane Spillback Time, h	never	never	never	never	never	never
1	ShrdLane Spillback Time, h	never	never	never	never	never	never
1	Base Free-Flow Speed, mph		40.78			40.78	
1	Running Time, s		33.48			33.48	
1	Running Speed, mph		36.65			36.65	
1	Through Delay, s/veh		20.862			20.862	
1	Travel Speed, mph		22.58			22.58	
1	Stop Rate, stops/veh		0.608			0.608	
1	Spatial Stop Rate, stops/mi		1.78			1.78	
1	Through vol/cap ratio		0.57			0.57	
1	Percent of Base FFS		55.4			55.4	
1	Level of Service		C			C	
1	Proportion Left Lanes		0.33			0.33	
1	Auto. Traveler Perception Score		2.56			2.56	

SPILLBACK TIME, h: never

The spillback check procedure computes the time of spillback for each of the internal movements. For turn movements, the bay/lane spillback time represents the time before the turn bay overflows. For through movements, the bay/lane spillback time represents the time before the through lane overflows due only to

through demand. If a turn bay exists and it overflows, then the turn volume will queue in the adjacent through lane. For this scenario, the shared lane spillback time is computed and used instead of the bay/lane spillback time. If several movements experience spillback, then the time of first spillback is reported at the bottom of Exhibit 17-38.

The output data for the two through movements are listed in Exhibit 17-38, starting with the third row. The base free-flow speed (FFS) and running time statistics are computed during Step 2: Determine Running Time. The through delay listed is computed during Step 5: Determine Through Control Delay. It represents a weighted average delay for the lane groups serving through movements at the downstream boundary intersection. The weight used in this average is the volume of through vehicles served by the lane group.

The percent of base free-flow speed equals the travel speed divided by the base free-flow speed. It and the through movement volume-to-capacity ratio are used with Exhibit 17-2 to determine that the segment is operating at LOS C in both travel directions.

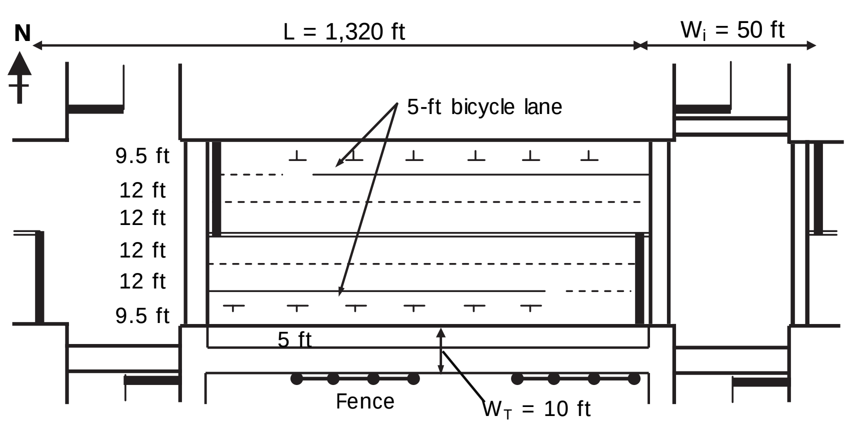
Each travel direction has one left-turn bay and three intersections. Thus, the proportion of intersections with left-turn lanes is 0.33. This proportion is used in Step 10: Determine Automobile Traveler Perception Score to compute the score of 2.56, which suggests that most automobile travelers would find segment service to be very good.

EXAMPLE PROBLEM 2: PEDESTRIAN LOS

The Segment

The sidewalk of interest is located along a 1,320-ft urban street segment. The segment is part of a collector street located near a community college. It is shown in Exhibit 17-39. Sidewalk is only shown for the south side of the segment for the convenience of illustration. It also exists on the north side of the segment.

Exhibit 17-39
Example Problem 2: Segment Geometry



The Question

What is the pedestrian LOS for the sidewalk on the south side of the segment?

The Facts

The geometric details of the sidewalk and street cross section are shown in Exhibit 17-39. Both boundary intersections are signalized. It is legal to cross the segment at uncontrolled, midsegment locations. The following additional information is known about the sidewalk and street segment:

Traffic Characteristics

Midsegment flow rate in eastbound direction: 940 veh/h

Pedestrian flow rate in south sidewalk (walking in both directions): 2,000 p/h

Proportion of on-street parking occupied during analysis period: 0.20

Geometric Characteristics

Shoulder width consists of 8.0 ft for parking and 1.5 ft for gutter pan.

Cross section has raised curb along outside edge of roadway.

Effective width of fixed objects on sidewalk: 0.0 ft (no objects present)

Presence of trees, bushes, or other vertical objects in buffer: No

Other Data

Pedestrians can cross the segment legally and do so somewhat uniformly along its length.

Proportion of sidewalk adjacent to window display: 0.0

Proportion of sidewalk adjacent to building face: 0.0

Proportion of sidewalk adjacent to fence: 0.50

Performance Measures Obtained from Supporting Methodologies

Motorized vehicle running speed: 33 mi/h

Pedestrian delay when walking parallel to the segment: 40 s/p

Pedestrian delay when crossing the segment at the nearest signal-controlled crossing: 80 s/p

Pedestrian waiting delay: 740 s/p

Pedestrian LOS score for the downstream intersection: 3.6

Outline of Solution

First, the pedestrian space will be calculated for the sidewalk. This measure will then be compared with the qualitative descriptions of pedestrian space listed in Exhibit 17-16. Next, the pedestrian travel speed along the sidewalk will be calculated. Finally, LOS for the crossing will be determined by using the computed pedestrian LOS score and pedestrian space variables with Exhibit 17-3.

Computational Steps

Step 1: Determine Free-Flow Walking Speed

The average free-flow walking speed is estimated to be 4.4 ft/s on the basis of the guidance provided.

Step 2: Determine Average Pedestrian Space

The shy distance on the inside of the sidewalk is computed by using Equation 17-23.

$$W_{s,i} = \max(W_{buf}, 1.5)$$

$$W_{s,i} = \max(5.0, 1.5)$$

$$W_{s,i} = 5.0 \text{ ft}$$

The shy distance on the outside of the sidewalk is computed by using Equation 17-24.

$$W_{s,o} = 3.0P_{\text{window}} + 2.0\, P_{\text{building}} + 1.5\, P_{\text{fence}}$$

$$W_{s,o} = 3.0(0.0) + 2.0\,(0.0) + 1.5\,(0.50)$$

$$W_{s,o} = 0.75 \text{ ft}$$

There are no fixed objects present on the sidewalk, so the adjusted fixed-object effective widths for the inside and outside of the sidewalk are both equal to 0.0 ft. The effective sidewalk width is computed by using Equation 17-22.

$$W_E = W_T - W_{O,i} - W_{O,o} - W_{s,i} - W_{s,o} \quad \geq 0.0$$

$$W_E = 10 - 0.0 - 0.0 - 5.0 - 0.75$$

$$W_E = 4.25 \text{ ft}$$

The pedestrian flow per unit width of sidewalk is computed by using Equation 17-27 for the subject sidewalk.

$$v_p = \frac{v_{ped}}{60\, W_E}$$

$$v_p = \frac{2{,}000}{60\,(4.25)}$$

$$v_p = 7.84 \text{ p/ft/min}$$

The average walking speed S_p is computed by using Equation 17-28.

$$S_p = (1 - 0.00078\, v_p^2)\, S_{pf} \quad \geq 0.5\, S_{pf}$$

$$S_p = (1 - 0.00078\, [7.84]^2)\, 4.4$$

$$S_p = 4.19 \text{ ft/s}$$

Finally, Equation 17-29 is used to compute average pedestrian space.

$$A_p = 60\, \frac{S_p}{v_p}$$

$$A_p = 60\, \frac{4.19}{7.84}$$

$$A_p = 32.0 \text{ ft}^2\text{/p}$$

The pedestrian space can be compared with the ranges provided in Exhibit 17-16 to make some judgments about the performance of the subject intersection corner. The criteria for platoon flow are considered applicable given the influence of the signalized intersections. According to the qualitative descriptions provided in this exhibit, walking speed will be restricted as will the ability to pass slower pedestrians.

Step 3: Determine Pedestrian Delay at Intersection

The pedestrian methodology in Chapter 18, Signalized Intersections, was used to estimate two pedestrian delay values. One value is the pedestrian delay at the boundary intersection when walking parallel to segment d_{pp}. This delay was computed to be 40 s/p. The second value is the pedestrian delay when crossing the segment at the nearest signal-controlled crossing d_{pc}. This delay was computed to be 80 s/p.

The pedestrian methodology in Chapter 19, Two-Way STOP-Controlled Intersections, was used to estimate the delay incurred while waiting for an acceptable gap in traffic d_{pw}. This delay was computed to be 740 s/p.

Step 4: Determine Pedestrian Travel Speed

The pedestrian travel speed is computed by using Equation 17-30.

$$S_{Tp,seg} = \frac{L}{\frac{L}{S_p} + d_{pp}}$$

$$S_{Tp,seg} = \frac{1{,}320}{\frac{1{,}320}{4.19} + 40}$$

$$S_{Tp,seg} = 3.72 \text{ ft/s}$$

This walking speed is slightly less than 4.0 ft/s and is considered acceptable, but a higher speed is desirable.

Step 5: Determine Pedestrian LOS Score for Intersection

The pedestrian methodology in Chapter 18 was used to determine the pedestrian LOS score for the downstream boundary intersection $I_{p,int}$. It was computed to be 3.60.

Step 6: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link is computed from three factors. However, before these factors can be calculated, several cross-section variables need to be adjusted and several coefficients need to be calculated. These variables and coefficients are calculated first. Then, the three factors are computed. Finally, they are combined to determine the desired score.

The total width of the outside through lane, bicycle lane, and paved shoulder W_t is computed as

$$W_t = W_{ol} + W_{bl}$$
$$W_t = 12 + 5$$
$$W_t = 17 \text{ ft}$$

In fact, the variable W_t does not include the width of the paved outside shoulder in this instance because the proportion of occupied on-street parking exceeds 0.0.

The effective total width of the outside through lane, bicycle lane, and shoulder as a function of traffic volume W_v is equal to W_t because the midsegment flow rate is greater than 160 veh/h.

The street cross section is curbed, so the adjusted width of paved outside shoulder W_{os}^* is 8.0 ft (= 9.5 – 1.5).

Because the proportion of occupied on-street parking is less than 0.25, the effective width of the combined bicycle lane and shoulder W_1 is computed as

$$W_1 = W_{bl} + W_{os}^*$$
$$W_1 = 5 + 8$$
$$W_1 = 13 \text{ ft}$$

The adjusted available sidewalk width W_{aA} is computed as

$$W_{aA} = \min(W_T - W_{buf}, 10)$$
$$W_{aA} = \min(10 - 5, 10)$$
$$W_{aA} = 5.0 \text{ ft}$$

The sidewalk width coefficient f_{sw} is computed as

$$f_{sw} = 6.0 - 0.3\, W_{aA}$$
$$f_{sw} = 6.0 - 0.3(5.0)$$
$$f_{sw} = 4.5 \text{ ft}$$

The buffer area coefficient f_b is equal to 1.0 because there is no continuous barrier at least 3.0 ft high located in the buffer area.

The automobile methodology described in Section 2 was used to determine the motorized vehicle running speed S_R for the subject segment. This speed was computed to be 33.0 mi/h.

The cross-section adjustment factor is computed by using Equation 17-32.

$$F_w = -1.2276 \ln(W_v + 0.5\, W_1 + 50\, p_{pk} + W_{buf}\, f_b + W_{aA}\, f_{sw})$$
$$F_w = -1.2276 \ln(17 + 0.5\,(13) + 50\,(0.20) + 5.0\,(1.0) + 5.0\,(4.5))$$
$$F_w = -5.05$$

The motorized vehicle volume adjustment factor is computed by using Equation 17-33.

$$F_v = 0.0091 \frac{v_m}{4\ N_{th}}$$

$$F_v = 0.0091 \frac{940}{4\ (2)}$$

$$F_v = 1.07$$

The motorized vehicle speed adjustment factor is computed by using Equation 17-34.

$$F_s = 4\left(\frac{S_R}{100}\right)^2$$

$$F_s = 4\left(\frac{33.0}{100}\right)^2$$

$$F_s = 0.44$$

Finally, the pedestrian LOS score for the link $I_{p,link}$ is calculated by using Equation 17-31.

$$I_{p,link} = 6.0468 + F_w + F_v + F_S$$

$$I_{p,link} = 6.0468 + (-5.05) + 1.07 + 0.44$$

$$I_{p,link} = 2.51$$

Step 7: Determine Link LOS

The pedestrian LOS for the link is determined by using the pedestrian LOS score from Step 6 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit 17-3 to determine that the LOS for the specified direction of travel along the subject link is C.

Step 8: Determine Roadway Crossing Difficulty Factor

Crossings occur somewhat uniformly along the length of the segment and the segment is bounded by two signalized intersections. Thus, the distance D_c is assumed to equal one-third of the segment length, or 440 ft (= 1,320/3), and the diversion distance D_d is computed as 880 ft (= 2 × 440 ft).

The delay incurred due to diversion is calculated by using Equation 17-36.

$$d_{pd} = \frac{D_d}{S_p} + d_{pc}$$

$$d_{pd} = \frac{880}{4.19} + 80$$

$$d_{pd} = 290 \text{ s/p}$$

The crossing delay used to estimate the roadway crossing difficulty factor is computed as

$$d_{px} = \min(d_{pd}, d_{pw}, 60)$$

$$d_{px} = \min(290, 740, 60)$$

$$d_{px} = 60 \text{ s/p}$$

The roadway crossing difficulty factor is computed by using Equation 17-37.

$$F_{cd} = 1.0 + \frac{0.10\ d_{px} - (0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606)}{7.5} \quad \leq 1.20$$

$$F_{cd} = 1.0 + \frac{0.10\ (60) - (0.318\ [2.51] + 0.220\ [3.60] + 1.606)}{7.5}$$

$$F_{cd} = 1.20$$

Step 9: Determine Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is computed by using Equation 17-38.

$$I_{p,seg} = F_{cd}\left(0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606\right)$$

$$I_{p,seg} = 1.20\ \left(0.318\ [2.51] + 0.220[3.60] + 1.606\right)$$

$$I_{p,seg} = 3.83$$

Step 10: Determine Segment LOS

The pedestrian LOS for the segment is determined by using the pedestrian LOS score from Step 9 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit 17-3 to determine that the LOS for the specified direction of travel along the subject segment is D.

EXAMPLE PROBLEM 3: BICYCLE LOS

The Segment

The bicycle lane of interest is located along a 1,320-ft urban street segment. The segment is part of a collector street located near a community college. The bicycle lane is provided for the eastbound direction of travel, as shown in Exhibit 17-40.

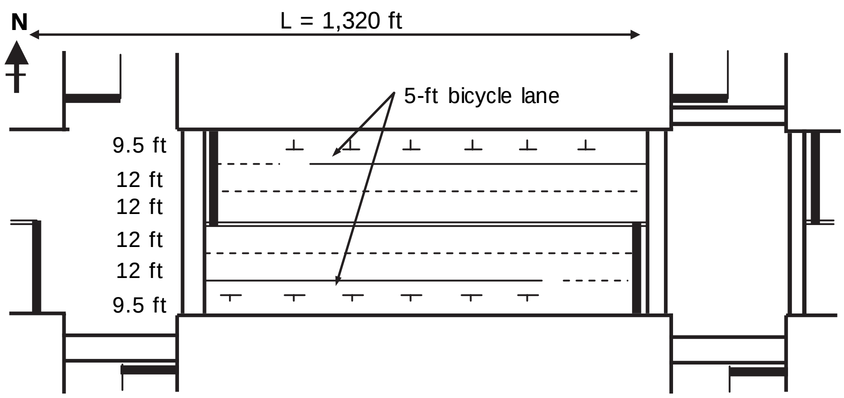


Exhibit 17-40
Example Problem 3: Segment Geometry

The Question

What is the bicycle LOS for the eastbound bicycle lane?

The Facts

The geometric details of the street cross section are shown in Exhibit 17-40. Both boundary intersections are signalized. The following additional information is known about the street segment:

<u>Traffic Characteristics</u>

Midsegment flow rate in eastbound direction: 940 veh/h

Percent heavy vehicles: 8.0%

Proportion of on-street parking occupied during analysis period: 0.20

<u>Geometric Characteristics</u>

Shoulder width consists of 8.0 ft for parking and 1.5 ft for gutter pan.

Cross section has raised curb along outside edge of roadway.

Number of access point approaches on right side of segment in subject travel direction: 3

<u>Other Data</u>

Pavement condition rating: 2.0

<u>Performance Measures Obtained from Supporting Methodologies</u>

Motorized vehicle running speed: 33 mi/h

Bicycle control delay: 40 s/bicycle

Bicycle LOS score for the downstream intersection: 0.08

Outline of Solution

First, the bicycle delay at the boundary intersection will be computed. This delay will then be used to compute the bicycle travel speed. Next, a bicycle LOS score will be computed for the link. It will then be combined with a similar score for the boundary intersection and used to compute the bicycle LOS score for the segment. Finally, LOS for the segment will be determined by using the computed score and the thresholds in Exhibit 17-4.

Computational Steps

Step 1: Determine Bicycle Running Speed

The average bicycle running speed S_b could not be determined from field data. Therefore, it was estimated to be 15 mi/h on the basis of the guidance provided.

Step 2: Determine Bicycle Delay at Intersection

The methodology in Chapter 18, Signalized Intersections, was used to estimate the bicycle delay at the boundary intersection d_b. This delay was computed to be 40.0 s/bicycle.

Step 3: Determine Bicycle Travel Speed

The segment running time of through bicycles is computed as

$$t_{Rb} = \frac{3{,}600\ L}{5{,}280\ S_b}$$

$$t_{Rb} = \frac{3{,}600\ (1{,}320)}{5{,}280\ (15)}$$

$$t_{Rb} = 60.0\ \text{s}$$

The average bicycle travel speed is computed by using Equation 17-39.

$$S_{Tb,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rb} + d_b)}$$

$$S_{Tb,seg} = \frac{3{,}600\ (1{,}320)}{5{,}280\ (60.0 + 40.0)}$$

$$S_{Tb,seg} = 9.0\ \text{mi/h}$$

This travel speed is adequate, but a speed of 10 mi/h or more is considered desirable.

Step 4: Determine Bicycle LOS Score for Intersection

The bicycle methodology in Chapter 18 was used to determine the bicycle LOS score for the boundary intersection $I_{b,int}$. It was computed to be 0.08.

Step 5: Determine Bicycle LOS Score for Link

The bicycle LOS score is computed from four factors. However, before these factors can be calculated, several cross-section variables need to be adjusted. These variables are calculated first, and then the four factors are computed. Finally, they are combined to determine the desired score.

The total width of the outside through lane, bicycle lane, and paved shoulder W_t is computed as

$$W_t = W_{ol} + W_{bl}$$

$$W_t = 12 + 5$$

$$W_t = 17 \text{ ft}$$

In fact, the variable W_t does not include the width of the paved outside shoulder in this instance because the proportion of occupied on-street parking exceeds 0.0.

The effective total width of the outside through lane, bicycle lane, and shoulder as a function of traffic volume W_v is equal to W_t because the midsegment flow rate is greater than 160 veh/h.

The street cross section is curbed, so the adjusted width of paved outside shoulder W_{os}^* is 8.0 ft (= 9.5 – 1.5).

Because the combined bicycle lane and adjusted shoulder width exceed 4.0 ft, the effective width of the outside through lane is computed as

$$W_e = W_v + W_{bl} + W_{os}^* - 20\, p_{pk} \quad \geq 0.0$$

$$W_e = 17 + 5 + 8 - 20\,(0.20)$$

$$W_e = 26 \text{ ft}$$

The percent heavy vehicles is less than 50%, so the adjusted percent heavy vehicles P_{HVa} is equal to the input percent heavy vehicles P_{HV} of 8.0%.

The automobile methodology described in Section 2 was used to determine the motorized vehicle running speed S_R for the subject segment. This speed was computed to be 33.0 mi/h. This speed exceeds 21 mi/h, so the adjusted motorized vehicle speed S_{Ra} is also equal to 33.0 mi/h.

The midsegment demand flow rate is greater than 8 veh/h (= 4 N_{th}), so the adjusted midsegment demand flow rate v_{ma} is equal to the input demand flow rate of 940 veh/h.

The cross-section adjustment factor is computed by using Equation 17-41.

$$F_w = -0.005\, W_e^2$$

$$F_w = -0.005\,(26)^2$$

$$F_w = -3.38$$

The motorized vehicle volume adjustment factor comes from Equation 17-42.

$$F_v = 0.507 \ln\left(\frac{v_{ma}}{4\, N_{th}}\right)$$

$$F_v = 0.507 \ln\left(\frac{940}{4\,(2)}\right)$$

$$F_v = 2.42$$

The motorized vehicle speed adjustment factor is computed by using Equation 17-43.

$$F_S = 0.199\left[1.1199\ln(S_{Ra} - 20) + 0.8103\right](1 + 0.1038\ P_{HVa})^2$$

$$F_S = 0.199\left[1.1199\ \ln(33.0 - 20) + 0.8103\right](1 + 0.1038\ (8.0))^2$$

$$F_S = 2.46$$

The pavement condition adjustment factor is computed by using Equation 17-44.

$$F_p = \frac{7.066}{P_c^{\,2}}$$

$$F_p = \frac{7.066}{2.0^2}$$

$$F_p = 1.77$$

Finally, the bicycle LOS score for the link $I_{b,link}$ is calculated by using Equation 17-40.

$$I_{b,link} = 0.760 + F_w + F_v + F_S + F_p$$

$$I_{b,link} = 0.760 - 3.38 + 2.42 + 2.46 + 1.77$$

$$I_{b,link} = 4.02$$

Step 6: Determine Link LOS

The bicycle LOS for the link is determined by using the bicycle LOS score from Step 5. This performance measure is compared with the thresholds in Exhibit 17-4 to determine that the LOS for the specified direction of travel along the subject link is D.

Step 7: Determine Bicycle LOS Score for Segment

The bicycle LOS score for the segment is computed by using Equation 17-45.

$$I_{b,seg} = 0.160\ I_{b,link} + 0.011\ F_{bi}\ e^{I_{b,int}} + 0.035\ \frac{N_{ap,s}}{(L/5{,}280)} + 2.85$$

$$I_{b,seg} = 0.160\ (4.02) + 0.011\ (1)\ e^{0.080} + 0.035\ \frac{3}{1{,}320/5{,}280} + 2.85$$

$$I_{b,seg} = 3.92$$

Step 8: Determine Segment LOS

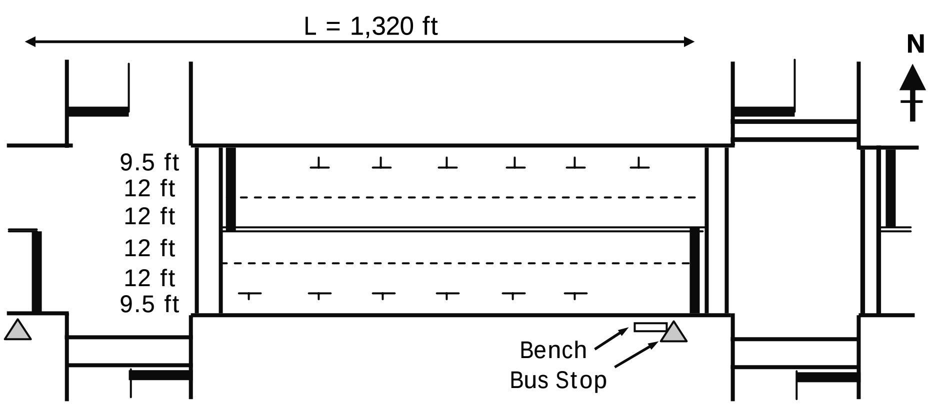
The bicycle LOS for the segment is determined by using the bicycle LOS score from Step 7. This performance measure is compared with the thresholds in Exhibit 17-4 to determine that the LOS for the specified direction of travel along the subject segment is D.

EXAMPLE PROBLEM 4: TRANSIT LOS

The Segment

The transit route of interest travels east along a 1,320-ft urban street segment. The segment is part of a collector street located near a community college. It is shown in Exhibit 17-41. A bus stop is provided on the south side of the segment for the subject route.

Exhibit 17-41
Example Problem 4: Segment Geometry



The Question

What is the transit LOS for the eastbound bus route while traveling the subject segment?

The Facts

The geometric details of the segment are shown in Exhibit 17-41. Both boundary intersections are signalized. There is one stop in the segment for the eastbound route. The following additional information is known about the bus stop and street segment:

<u>Transit Characteristics</u>

Dwell time: 20.0 s

Transit frequency: 4 veh/h

Excess wait time data are not available for the stop, but the on-time performance of the route (based on a standard of up to 5 min late being considered "on time") at the previous time point is known (92%).

Passenger load factor: 0.83 passengers/seat

<u>Other Data</u>

Area type: not in a central business district

The bus stop in the segment has a bench, but no shelter.

Number of routes serving the segment: 1

The bus stop is accessed from the right-turn lane (i.e., the stop is off-line). Buses are exempt from the requirement to turn right but have no other traffic priority.

Performance Measures Obtained from Supporting Methodologies

Motorized vehicle running speed: 33 mi/h

Pedestrian LOS score for the link: 3.53

Through control delay at downstream boundary intersection: 20.9 s/veh

Reentry delay: 16.17 s

g/*C* ratio at downstream boundary intersection: 0.4729

Outline of Solution

First, the transit vehicle segment running time will be computed. Next, the control delay at the boundary intersection will be obtained and used to compute the transit vehicle segment travel speed. Then, the transit wait–ride score will be computed. This score will be combined with the pedestrian LOS score for the link to compute the transit LOS score for the segment. Finally, LOS for the segment will be determined by comparing the computed score with the thresholds identified in Exhibit 17-4.

Computational Steps

Step 1: Determine Transit Vehicle Running Time

The transit vehicle running time is based on the segment running speed and delay due to a transit vehicle stop. These components are calculated first, and then running time is calculated.

Transit vehicle segment running speed can be computed by using Equation 17-46.

$$S_{Rt} = \min\left(S_R, \frac{61}{1+e^{-1.00+(1{,}185\,N_{ts}/L)}}\right)$$

$$S_{Rt} = \min\left(33.0, \frac{61}{1+e^{-1.00+(1{,}185\,[1]/1{,}320)}}\right)$$

$$S_{Rt} = 32.1 \text{ mi/h}$$

The acceleration and deceleration rates are unknown, so they are assumed to equal 4.0 ft/s^2.

The bus stop is located on the near side of a signalized intersection. From Equation 17-48, the average proportion of bus stop acceleration–deceleration delay not due to the intersection's traffic control f_{ad} is equal to the g/C ratio for the through movement in the bus's direction of travel (in this case, eastbound). The effective green time g is 47.29 s (calculated as the phase duration minus the change period), and the cycle length is 100 s. Therefore, f_{ad} is 0.4729.

Equation 17-47 can now be used to compute the portion of bus stop delay due to acceleration and deceleration.

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{S_{Rt}}{2}\right)\left(\frac{1}{r_{at}} + \frac{1}{r_{dt}}\right)f_{ad}$$

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{33.0}{2}\right)\left(\frac{1}{4.0} + \frac{1}{4.0}\right)(0.4729)$$

$$d_{ad} = 5.56 \text{ s}$$

Equation 17-49 is used to compute the portion of bus stop delay due to serving passengers, using the input average dwell time of 20.0 s and an f_{dt} value of 0.4729, based on the stop's near-side location at a traffic signal and the g/C ratio computed in a previous step. The f_{dt} factor is used to avoid double-counting the portion of passenger service time that occurs during the signal's red indication and is therefore included as part of control delay.

$$d_{ps} = t_d f_{dt}$$

$$d_{ps} = (20.0)(0.4729)$$

$$d_{ps} = 9.46 \text{ s}$$

The bus stop is located in the right-turn lane; therefore, the bus is subject to reentry delay upon leaving the stop. On the basis of the guidance for reentry delay for a near-side stop at a traffic signal, the reentry delay d_{re} is equal to the queue service time g_s. By following the procedures given in Chapter 18, Signalized Intersections, this time is calculated to be 16.17 s.

Equation 17-50 is used to compute the total delay due to the transit stop.

$$d_{ts} = d_{ad} + d_{ps} + d_{re}$$

$$d_{ts} = 5.56 + 9.46 + 16.17 = 31.19 \text{ s}$$

Equation 17-51 is used to compute transit vehicle running time on the basis of the previously computed components.

$$t_{Rt} = \frac{3{,}600\ L}{5{,}280\ S_{Rt}} + \sum_{i=1}^{N_{ts}} d_{ts,i}$$

$$t_{Rt} = \frac{3{,}600\ (1{,}320)}{5{,}280\ (32.1)} + 31.19$$

$$t_{Rt} = 59.3 \text{ s}$$

Step 2: Determine Delay at Intersection

The automobile control delay d at the boundary intersection was computed to be 20.9 s/veh.

Step 3: Determine Travel Speed

The average transit travel speed is computed by using Equation 17-53.

$$S_{Tt,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rt} + d)}$$

$$S_{Tt,seg} = \frac{3{,}600\,(1{,}320)}{5{,}280\,(59.3 + 20.9)}$$

$$S_{Tt,seg} = 11.2 \text{ mi/h}$$

Step 4: Determine Transit Wait–Ride Score

The wait–ride score is based on the headway factor and the perceived travel time factor. Each of these components is calculated separately. The wait–ride score is then calculated.

The input data indicate that there is one route on the segment, and its frequency is 4 veh/h. The headway factor is computed by using Equation 17-54.

$$F_h = 4.00\, e^{-1.434/(v_s + 0.001)}$$

$$F_h = 4.00\, e^{-1.434/(4 + 0.001)}$$

$$F_h = 2.80$$

The perceived travel time factor is based on several intermediate variables that need to be calculated first. The first of these calculations is the amenity time rate. It is calculated by using Equation 17-58. A default passenger trip length of 3.7 mi is used in the absence of other information.

$$T_{at} = \frac{1.3\, p_{sh} + 0.2\, p_{be}}{L_{pt}}$$

$$T_{at} = \frac{1.3\,(0.0) + 0.2\,(1.0)}{3.7} = 0.054 \text{ min/mi}$$

Since no information is available for actual excess wait time, but on-time performance information is available for the route, Equation 17-59 is used to estimate excess wait time.

$$t_{ex} = [t_{late}(1 - p_{ot})]^2$$

$$t_{ex} = [5.0(1 - 0.92)]^2$$

$$t_{ex} = 0.16 \text{ min}$$

The excess wait time rate T_{ex} is then the excess wait time t_{ex} divided by the average passenger trip length L_{pt}: 0.16/3.7 = 0.043 min/mi.

The passenger load waiting factor is computed by using Equation 17-57.

$$a_1 = 1 + \frac{(4)(F_l - 0.80)}{4.2}$$

$$a_1 = 1 + \frac{(4)(0.83 - 0.80)}{4.2}$$

$$a_1 = 1.03$$

The perceived travel time rate is computed by using Equation 17-56.

$$T_{ptt} = \left(a_1 \frac{60}{S_{Tt,seg}} \right) + \left(2\, T_{ex} \right) - T_{at}$$

$$T_{ptt} = \left(1.03 \frac{60}{11.2} \right) + \left(2\, [0.043] \right) - 0.054$$

$$T_{ptt} = 5.53 \text{ min/mi}$$

The segment is not located in a central business district of a metropolitan area with a population of 5 million or more, so the base travel time rate T_{btt} is equal to 4.0 min/mi. The perceived travel time factor is computed by using Equation 17-55.

$$F_{tt} = \frac{(e-1)\, T_{btt} - (e+1)\, T_{ptt}}{(e-1)\, T_{ptt} - (e+1)\, T_{btt}}$$

$$F_{tt} = \frac{(-0.40-1)\,(4.0) - (-0.40+1)(5.53)}{(-0.40-1)\,(5.53) - (-0.40+1)\,(4.0)}$$

$$F_{tt} = 0.88$$

Finally, the transit wait–ride score is computed by using Equation 17-60.

$$s_{w-r} = F_h\, F_{tt}$$

$$s_{w-r} = (2.80)(0.88)$$

$$s_{w-r} = 2.46$$

Step 5: Determine Pedestrian LOS Score for Link

The pedestrian methodology described in Section 2 was used to determine the pedestrian LOS score for the link $I_{p,link}$. This score was computed to be 3.53.

Step 6: Determine Transit LOS Score for Segment

The transit LOS score for the segment is computed by using Equation 17-61.

$$I_{t,seg} = 6.0 - 1.50\, s_{w-r} + 0.15\, I_{p,link}$$

$$I_{t,seg} = 6.0 - 1.50\,(2.46) + 0.15\,(3.53)$$

$$I_{t,seg} = 2.84$$

Step 7: Determine LOS

The transit LOS is determined by using the transit LOS score from Step 6. This performance measure is compared with the thresholds in Exhibit 17-4 to determine that the LOS for the specified bus route is C.

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CHAPTER 18
SIGNALIZED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 18, Signalized Intersections, describes a methodology for evaluating the capacity and quality of service provided to road users traveling through a signalized intersection. However, the methodology is much more than just a tool for evaluating capacity and quality of service. It includes an array of performance measures that describe intersection operation for multiple travel modes. These measures serve as clues for identifying the source of problems and provide insight into the development of effective improvement strategies. The analyst using this methodology is encouraged to consider the full range of measures.

OVERVIEW OF THE METHODOLOGY

This chapter's methodology applies to three- and four-leg intersections of two streets or highways where the signalization operates in isolation from nearby intersections.

The influence of an upstream signalized intersection on the subject intersection's operation is addressed by input variables that describe platoon structure and the uniformity of arrivals on a cyclic basis. Chapter 17, Urban Street Segments, describes a methodology for evaluating an intersection that is part of a coordinated signal system.

Analysis Boundaries

The intersection analysis boundaries are not defined at a fixed distance for all intersections. Rather, they are dynamic and extend backward from the intersection a sufficient distance to include the operational influence area on each intersection leg. The size of this area is leg-specific and includes the most distant extent of any intersection-related queue expected to occur during the study period. For these reasons, the analysis boundaries should be established for each intersection according to conditions during the analysis period. The influence area should extend at least 250 ft back from the stop line on each intersection leg.

Analysis Level

Analysis level describes the level of detail used when the methodology is applied. Three levels are recognized:

- Operational,
- Design, and
- Planning and preliminary engineering.

The operational analysis is the most detailed application and requires the most information about traffic, geometric, and signalization conditions. The design analysis also requires detailed information about traffic conditions and the desired level of service (LOS) as well as information about geometric or signalization conditions. The design analysis then seeks to determine reasonable values for the conditions not provided. The planning and preliminary engineering analysis requires only the most fundamental types of information

from the analyst. Default values are then used as substitutes for other input data. Analysis level is discussed in more detail in the applications section of this chapter.

Study Period and Analysis Period

The study period is the time interval represented by the performance evaluation. It consists of one or more consecutive analysis periods. An analysis period is the time interval evaluated by a single application of the methodology.

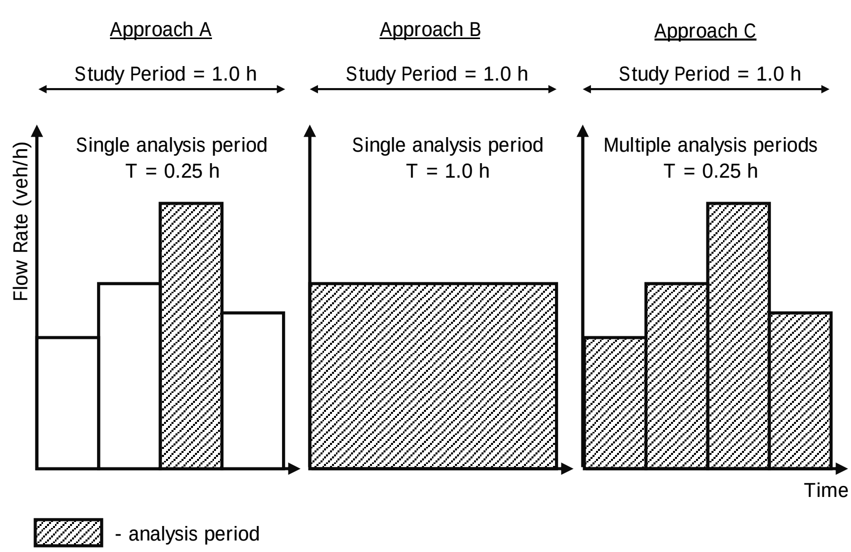
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the analysis period ranges from 0.25 to 1 h. The longer durations are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1 h because traffic conditions typically are not steady for long time periods and because the adverse impact of short peaks in traffic demand may not be detected in the evaluation.

If an analysis period of interest has a demand volume that exceeds capacity, then the study period should include an initial analysis period with no initial queue and a final analysis period with no residual queue. This approach provides a more accurate estimate of the delay associated with the congestion.

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be reported separately. In this situation, reporting an average performance for the study period is not encouraged because it may obscure extreme values and suggest acceptable operation when some analysis periods have unacceptable operation.

Exhibit 18-1 demonstrates three alternative approaches an analyst might use for a given evaluation. Other alternatives exist, and the study period can exceed 1 h. Approach A has traditionally been used and, unless otherwise justified, is the one recommended for use.

Exhibit 18-1
Three Alternative Study Approaches



Approach A is based on evaluation of the peak 15-min period during the study period. The analysis period T is 0.25 h. The equivalent hourly flow rate in

vehicles per hour (veh/h) used for the analysis is based on either a peak 15-min traffic count multiplied by four or a 1-h demand volume divided by the peak hour factor. The former option is preferred when traffic counts are available. Additional discussion on use of the peak hour factor is provided in the required input data subsection.

Approach B is based on evaluation of one 1-h analysis period that is coincident with the study period. The analysis period *T* is 1.0 h. The flow rate used is equivalent to the 1-h demand volume (i.e., the peak hour factor is not used). This approach implicitly assumes that the arrival rate of vehicles is constant throughout the period of study. Therefore, the effects of peaking within the hour may not be identified and the analyst risks underestimating the delay actually incurred.

Approach C uses a 1-h study period and divides it into four 0.25-h analysis periods. This approach accounts for systematic flow rate variation among analysis periods. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay.

Performance Measures

An intersection's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road user group. Performance measures cited in this chapter include automobile volume-to-capacity ratio, automobile delay, queue storage ratio, pedestrian delay, pedestrian circulation area, pedestrian perception score, bicycle delay, and bicycle perception score.

LOS is also considered a performance measure. It is computed for the automobile, pedestrian, and bicycle travel modes. It is useful for describing intersection performance to elected officials, policy makers, administrators, and the public. LOS is based on one or more of the performance measures listed in the preceding paragraph.

Travel Modes

This chapter describes three methodologies that can be used to evaluate intersection performance from the perspective of motorists, pedestrians, and bicyclists. They are referred to as the automobile methodology, the pedestrian methodology, and the bicycle methodology.

The automobile methodology has evolved and reflects the findings from a large body of research. It was originally based, in part, on the results of a National Cooperative Highway Research Program (NCHRP) study (*1*, *2*) that formalized the critical movement analysis procedure and the automobile delay estimation procedure. The critical movement analysis procedure was developed in the United States (*3*, *4*), Australia (*5*), Great Britain (*6*), and Sweden (*7*). The automobile delay estimation procedure was developed in Great Britain (*8*), Australia (*9*), and the United States (*10*). Updates to the original methodology were developed in a series of research projects (*11*–*24*). The procedures for evaluating pedestrian and bicyclist perception of LOS are documented in an NCHRP report (*25*). The procedures for evaluating pedestrian delay, pedestrian

circulation area, and bicyclist delay are documented in two Federal Highway Administration reports (*26, 27*).

The phrase *automobile mode*, as used in this chapter, refers to travel by all motorized vehicles that can legally operate on the street, with the exception of local transit vehicles that stop to pick up passengers at the intersection. Unless explicitly stated otherwise, the word *vehicles* refers to motorized vehicles and includes a mixed stream of automobiles, motorcycles, trucks, and buses.

Lane Groups and Movement Groups

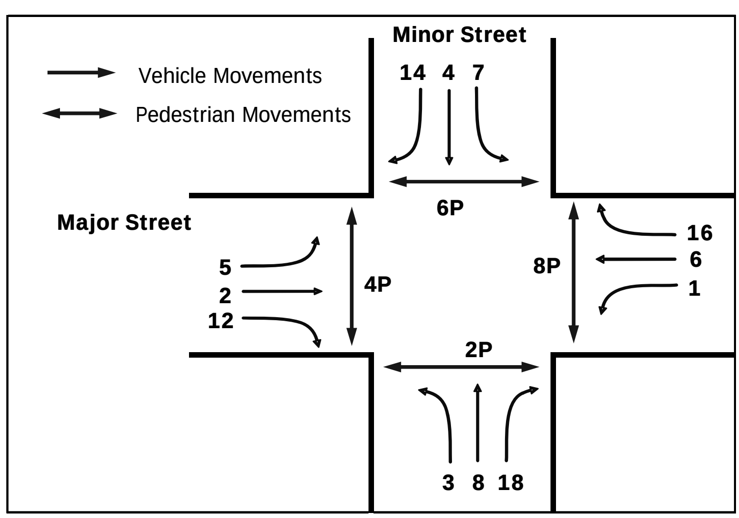
The automobile methodology is designed to evaluate the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group*. In general, a separate lane group is established for (*a*) each lane (or combination of adjacent lanes) that exclusively serves one movement and (*b*) each lane shared by two or more movements. Guidelines for establishing lane groups are described in Section 2, Methodology.

The concept of *movement groups* is also established to facilitate data entry. A separate movement group is established for (*a*) each turn movement with one or more exclusive turn lanes and (*b*) the through movement (inclusive of any turn movements that share a lane).

Movement and Phase Numbering

Exhibit 18-2 illustrates the vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. To facilitate the discussion in this chapter, each movement is assigned a unique number or a number and letter combination. The letter P denotes a pedestrian movement.

Exhibit 18-2
Intersection Traffic Movements and Numbering Scheme

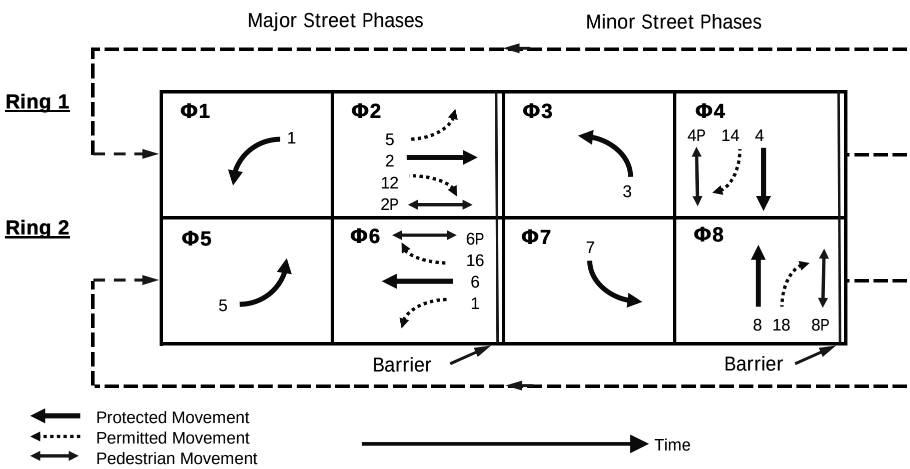


Modern actuated controllers implement signal phasing by using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other. The commonly used eight-phase dual-ring structure is shown in Exhibit

18-3. The symbol Φ shown in this exhibit represents the word "phase," and the number following the symbol represents the phase number.

Exhibit 18-3 shows one way that traffic movements can be assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is assigned to an exclusive phase. During this phase, the left-turn movement is "protected" so that it receives a green arrow indication. Each through, right-turn, and pedestrian movement combination is also assigned to an exclusive phase. The dashed arrows indicate turn movements that are served in a "permitted" manner so that the turn can be completed only after yielding the right-of-way to conflicting movements. Additional information about traffic signal controller operation is provided in Chapter 31, Signalized Intersections: Supplemental.

Exhibit 18-3
Dual-Ring Structure with Illustrative Movement Assignments



LOS CRITERIA

This subsection describes the LOS criteria for the automobile, pedestrian, and bicycle modes. The criteria for the automobile mode are different from those for the nonautomobile modes. Specifically, the automobile-mode criteria are based on performance measures that are field measurable and perceivable by travelers. The criteria for the nonautomobile modes are based on scores reported by travelers indicating their perception of service quality.

Automobile Mode

All uses of the word "volume" or the phrase "volume-to-capacity ratio" in this chapter refer to demand volume or demand-volume-to-capacity ratio.

LOS can be characterized for the entire intersection, each intersection approach, and each lane group. Control delay alone is used to characterize LOS for the entire intersection or an approach. Control delay *and* volume-to-capacity ratio are used to characterize LOS for a lane group. Delay quantifies the increase in travel time due to traffic signal control. It is also a surrogate measure of driver discomfort and fuel consumption. The volume-to-capacity ratio quantifies the degree to which a phase's capacity is utilized by a lane group. The following paragraphs describe each LOS.

LOS A describes operations with a control delay of 10 s/veh or less and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is low and either progression is exceptionally

favorable or the cycle length is very short. If it is due to favorable progression, most vehicles arrive during the green indication and travel through the intersection without stopping.

LOS B describes operations with control delay between 10 and 20 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is low and either progression is highly favorable or the cycle length is short. More vehicles stop than with LOS A.

LOS C describes operations with control delay between 20 and 35 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when progression is favorable or the cycle length is moderate. Individual *cycle failures* (i.e., one or more queued vehicles are not able to depart as a result of insufficient capacity during the cycle) may begin to appear at this level. The number of vehicles stopping is significant, although many vehicles still pass through the intersection without stopping.

LOS D describes operations with control delay between 35 and 55 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is high and either progression is ineffective or the cycle length is long. Many vehicles stop and individual cycle failures are noticeable.

LOS E describes operations with control delay between 55 and 80 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is high, progression is unfavorable, and the cycle length is long. Individual cycle failures are frequent.

LOS F describes operations with control delay exceeding 80 s/veh or a volume-to-capacity ratio greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is very high, progression is very poor, and the cycle length is long. Most cycles fail to clear the queue.

A lane group can incur a delay less than 80 s/veh when the volume-to-capacity ratio exceeds 1.0. This condition typically occurs when the cycle length is short, the signal progression is favorable, or both. As a result, both the delay and volume-to-capacity ratio are considered when lane group LOS is established. A ratio of 1.0 or more indicates that cycle capacity is fully utilized and represents failure from a capacity perspective (just as delay in excess of 80 s/veh represents failure from a delay perspective).

Exhibit 18-4 lists the LOS thresholds established for the automobile mode at a signalized intersection.

Exhibit 18-4
LOS Criteria: Automobile Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a]	
	≤1.0	>1.0
≤10	A	F
>10–20	B	F
>20–35	C	F
>35–55	D	F
>55–80	E	F
>80	F	F

Note: [a] For approach-based and intersectionwide assessments, LOS is defined solely by control delay.

Nonautomobile Modes

Historically, the HCM has used a single performance measure as the basis for defining LOS. However, research documented in Chapter 5, Quality and Level-of-Service Concepts, indicates that travelers consider a wide variety of factors in assessing the quality of service provided to them. Some of these factors can be described as performance measures (e.g., speed) and others can be described as basic descriptors of the intersection character (e.g., crosswalk width). The methodology for evaluating each mode provides a procedure for mathematically combining these factors into a score. This score is then used to determine the LOS that is provided.

Exhibit 18-5 lists the range of scores associated with each LOS for the pedestrian and bicycle travel modes. The association between score value and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip through a signalized intersection. The letter A was used to represent the best quality of service, and the letter F was used to represent the worst quality of service. “Best” and “worst” were left undefined, allowing respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

Exhibit 18-5
LOS Criteria: Pedestrian and Bicycle Modes

LOS	LOS Score
A	≤2.00
B	>2.00–2.75
C	>2.75–3.50
D	>3.50–4.25
E	>4.25–5.00
F	>5.00

REQUIRED INPUT DATA

This subsection describes the required input data for the automobile, pedestrian, and bicycle methodologies. Default values for some of these data are provided in Section 3, Applications.

Automobile Mode

This part describes the input data needed for the automobile methodology. The data needed for fully or semiactuated signal control are listed in Exhibit 18-6. The additional data needed for coordinated-actuated control are listed in Exhibit 18-7.

The last column of Exhibit 18-6 and Exhibit 18-7 indicates whether the input data are needed for each traffic movement, a specific movement group, each signal phase, each intersection approach, or the intersection as a whole.

The data elements listed in Exhibit 18-6 and Exhibit 18-7 do not include variables that are considered to represent calibration factors (e.g., start-up lost time). Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Exhibit 18-6
Input Data Requirements: Automobile Mode with Pretimed, Fully Actuated, or Semiactuated Signal Control

Data Category	Input Data Element	Basis
Traffic characteristics	Demand flow rate	Movement
	Right-turn-on-red flow rate	Approach
	Percent heavy vehicles	Movement group
	Intersection peak hour factor	Intersection
	Platoon ratio	Movement group
	Upstream filtering adjustment factor	Movement group
	Initial queue	Movement group
	Base saturation flow rate	Movement group
	Lane utilization adjustment factor	Movement group
	Pedestrian flow rate	Approach
	Bicycle flow rate	Approach
	On-street parking maneuver rate	Movement group
	Local bus stopping rate	Approach
Geometric design	Number of lanes	Movement group
	Average lane width	Movement group
	Number of receiving lanes	Approach
	Turn bay length	Movement group
	Presence of on-street parking	Movement group
	Approach grade	Approach
Signal control	Type of signal control	Intersection
	Phase sequence	Intersection
	Left-turn operational mode	Approach
	Dallas left-turn phasing option	Approach
	Passage time (if actuated)	Phase
	Maximum green (or green duration if pretimed)	Phase
	Minimum green	Phase
	Yellow change	Phase
	Red clearance	Phase
	Walk	Phase
	Pedestrian clear	Phase
	Phase recall	Phase
	Dual entry (if actuated)	Phase
	Simultaneous gap-out (if actuated)	Approach
Other	Analysis period duration	Intersection
	Speed limit	Approach
	Stop-line detector length and detection mode	Movement group
	Area type	Intersection

Notes: Movement = one value for each left-turn, through, and right-turn movement.
Movement group = one value for each turn movement with exclusive turn lanes and one value for the through movement (inclusive of any turn movements in a shared lane).
Approach = one value or condition for the intersection approach.
Intersection = one value or condition for the intersection.
Phase = one value or condition for each signal phase.

Exhibit 18-7
Input Data Requirements: Automobile Mode with Coordinated-Actuated Signal Control

Data Category	Input Data Element	Basis
Signal control	Cycle length	Intersection
	Phase splits	Phase
	Offset	Intersection
	Offset reference point	Intersection
	Force mode	Intersection

Notes: Intersection = one value or condition for the intersection.
Phase = one value or condition for each signal phase.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 18-6. These data describe the motorized vehicle traffic stream that travels through the intersection during the study period.

Demand Flow Rate

The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. Demand flow rate represents the flow rate of vehicles *arriving* at the intersection. When measured in the field, this flow rate is based on a traffic count taken upstream of the queue associated with the subject intersection. This distinction is important for counts during congested periods because the count of vehicles departing from a congested approach will produce a demand flow rate that is lower than the true rate.

There is one exception to the aforementioned definition of demand flow rate. Specifically, if a planning analysis is being conducted where (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand can be divided by the intersection peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor should be based on local traffic peaking trends. If a local factor is not available, then the default value provided in Section 3 can be used.

In summary, demand flow rate for the analysis period is an input to the methodology. This rate is computed as the count of vehicles arriving during the period divided by the length of the period, expressed as an hourly flow rate, and without the use of a peak hour factor. If a peak hour factor is used, it must be used to compute the hourly flow rate that is input to the methodology.

If intersection operation is being evaluated during multiple sequential analysis periods, then the count of vehicles arriving during each analysis period should be provided for each movement.

The methodology includes a procedure for determining the distribution of flow among the available lanes on an approach with one or more shared lanes. The procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection, where the lane volume-to-saturation flow ratio is used to estimate relative differences in this time among lanes. This assumption may not always hold for situations in which drivers choose a lane so that they are prepositioned for a turn at the downstream intersection. In this situation, the analyst needs to provide the flow rate for each lane on the approach and then combine these rates to define explicitly the flow rate for each lane group.

Only right turns that are controlled by the signal should be represented in the right-turn volume input to the automobile methodology.

If a right-turn movement is allowed to turn right on the red indication, the analyst may reduce the right-turn flow rate by the flow rate of right-turn-on-red

(RTOR) vehicles. This topic is discussed in more detail in the next few paragraphs.

Right-Turn-on-Red Flow Rate

The RTOR flow rate is defined as the count of vehicles that turn right at the intersection when the controlling signal indication is red, divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h.

It is difficult to predict the RTOR flow rate because it is based on many factors that vary widely from intersection to intersection. These factors include the following:

- Approach lane allocation (shared or exclusive right-turn lane),
- Right-turn flow rate,
- Sight distance available to right-turning drivers,
- Volume-to-capacity ratio for conflicting movements,
- Arrival patterns of right-turning vehicles during the signal cycle,
- Departure patterns of conflicting movements,
- Left-turn signal phasing on the conflicting street, and
- Conflicts with pedestrians.

Given the difficulty of estimating the RTOR flow rate, it should be measured in the field when possible. If the analysis is dealing with future conditions or if the RTOR flow rate is not known from field data, then the RTOR flow rate for each right-turn movement should be assumed to equal 0 veh/h. This assumption is conservative because it yields a slightly larger estimate of delay than may actually be incurred by intersection movements.

If the right-turn movement is served by an exclusive lane and a complementary left-turn phase exists on the cross street, then the right-turn volume for analysis can be reduced by the number of shadowed left turners (with both movements being considered on an equivalent, per lane basis).

Percent Heavy Vehicles

A heavy vehicle is defined as any vehicle with more than four tires touching the pavement. Local buses that stop within the intersection area are not included in the count of heavy vehicles. The percentage of heavy vehicles represents the count of heavy vehicles that arrive during the analysis period divided by the total vehicle count for the same period. This percentage is provided for each intersection traffic movement; however, one representative value for all movements may be used for a planning analysis.

Intersection Peak Hour Factor

One peak hour factor for the entire intersection is computed with the following equation:

$$PHF = \frac{n_{60}}{4\, n_{15}}$$

Equation 18-1

where

PHF = peak hour factor,

n_{60} = count of vehicles during a 1-h period (veh), and

n_{15} = count of vehicles during the peak 15-min period (veh).

The count used in the denominator of Equation 18-1 must be taken during a 15-min period that occurs within the 1-h period represented by the variable in the numerator. Both variables in this equation represent the total number of vehicles entering the intersection during their respective time period. As such, one peak hour factor is computed for the intersection. This factor is then applied individually to each traffic movement. Values of this factor typically range from 0.80 to 0.95.

As noted previously, the peak hour factor is used primarily for a planning analysis when a forecast hourly volume is provided and an analysis of the peak 15-min period is sought. Normally, the demand flow rate is computed as the count of vehicles arriving during the period divided by the length of the period, expressed as an hourly flow rate, and without the use of a peak hour factor.

The use of a single peak hour factor for the entire intersection is intended to avoid the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, whereas in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. In the event that individual approaches or movements are known to peak at different times, several 15-min analysis periods that encompass all the peaking should be considered instead of a single analysis in which all the peak hour factors are used together, as if the peaks they represent also occurred together.

Platoon Ratio

Platoon ratio is used to describe the quality of signal progression for the corresponding movement group. It is computed as the demand flow rate during the green indication divided by the average demand flow rate. Values for the platoon ratio typically range from 0.33 to 2.0. Exhibit 18-8 provides an indication of the quality of progression associated with selected platoon ratio values.

Exhibit 18-8
Relationship Between Arrival Type and Progression Quality

Platoon Ratio	Arrival Type	Progression Quality
0.33	1	Very poor
0.67	2	Unfavorable
1.00	3	Random arrivals
1.33	4	Favorable
1.67	5	Highly favorable
2.00	6	Exceptionally favorable

For protected or protected-permitted left-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the associated turn phase (i.e., the protected period). Hence, the platoon ratio is based on the flow rate during the green indication of the left-turn phase.

For permitted left-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the permitted period. Hence, the platoon ratio is based on the left-turn flow rate during the green indication of the phase providing the permitted operation.

For permitted or protected-permitted right-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the permitted period (even if a protected right-turn operation is provided during the complementary left-turn phase on the cross street). Hence, the platoon ratio is based on the right-turn flow rate during the green indication of the phase providing the permitted operation.

For through movements served by exclusive lanes (no shared lanes on the approach), the platoon ratio for the through movement group is based on the through flow rate during the green indication of the associated phase.

For all movements served by split phasing, the platoon ratio for a movement group is based on its flow rate during the green indication of the common phase.

For intersection approaches with one or more shared lanes, one platoon ratio is computed for the shared movement group on the basis of the flow rate of all shared lanes (plus that of any exclusive through lanes that are also served) during the green indication of the common phase.

The platoon ratio for a movement group can be estimated from field data with the following equation:

Equation 18-2

$$R_p = \frac{P}{(g/C)}$$

where

R_p = platoon ratio,

P = proportion of vehicles arriving during the green indication (decimal),

g = effective green time (s), and

C = cycle length (s).

The "proportion of vehicles arriving during the green indication" P is computed as the count of vehicles that arrive during the green indication divided by the count of vehicles that arrive during the entire signal cycle. It is an average value representing conditions during the analysis period.

If the subject intersection is part of a signal system, then the procedure in Chapter 17, Urban Street Segments, can be used to estimate the arrival flow profile for any approach that is evaluated as part of an urban street segment. The procedure uses the profile to compute the proportion of arrivals during the green indication. If this procedure is used, then platoon ratio is not an input for the traffic movements on the subject approach.

If the subject intersection is not part of a signal system and an existing intersection is being evaluated, then it is recommended that analysts use field-measured values for the variables in Equation 18-2 in estimating the platoon ratio.

If the subject intersection is not part of a signal system and the analysis is dealing with future conditions, or if the variables in Equation 18-2 are not known from field data, then the platoon ratio can be judged from Exhibit 18-8 by using the arrival type designation. Values of arrival type range from 1 to 6. A description of each arrival type is provided in the following paragraphs to help the analyst make a selection.

Arrival Type 1 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the red interval. This arrival type is often associated with short segments with very poor progression in the subject direction of travel (and possibly good progression for the other direction).

Arrival Type 2 is characterized by a moderately dense platoon arriving in the middle of the red interval or a dispersed platoon containing 40% to 80% of the movement group volume arriving throughout the red interval. This arrival type is often associated with segments of average length with unfavorable progression in the subject direction of travel.

Arrival Type 3 describes one of two conditions. If the signals bounding the segment are coordinated, then this arrival type is characterized by a platoon containing less than 40% of the movement group volume arriving partly during the red interval and partly during the green interval. If the signals are not coordinated, then this arrival type is characterized by platoons arriving at the subject intersection at different points in time over the course of the analysis period so that arrivals are effectively random.

Arrival Type 4 is characterized by a moderately dense platoon arriving in the middle of the green interval or a dispersed platoon containing 40% to 80% of the movement group volume arriving throughout the green interval. This arrival type is often associated with segments of average length with favorable progression in the subject direction of travel.

Arrival Type 5 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the green interval. This arrival type is often associated with short segments with highly favorable progression in the subject direction of travel and a low-to-moderate number of side street entries.

Arrival Type 6 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the green interval. This arrival type occurs only on very short segments with exceptionally favorable

progression in the subject direction of travel and negligible side street entries. It is reserved for routes in dense signal networks, possibly with one-way streets.

Upstream Filtering Adjustment Factor

The upstream filtering adjustment factor I accounts for the effect of an upstream signal on vehicle arrivals to the subject movement group. Specifically, this factor reflects the way an upstream signal changes the variance in the number of arrivals per cycle. The variance decreases with increasing volume-to-capacity ratio, which can reduce cycle failure frequency and resulting delay.

The filtering adjustment factor varies from 0.09 to 1.0. A value of 1.0 is appropriate for an isolated intersection (i.e., one that is 0.6 mi or more from the nearest upstream signalized intersection). A value of less than 1.0 is appropriate for nonisolated intersections. The following equation is used to compute I for nonisolated intersections:

Equation 18-3

$$I = 1.0 - 0.91\, X_u^{2.68} \quad \geq 0.090$$

where

I = upstream filtering adjustment factor, and

X_u = weighted volume-to-capacity ratio for all upstream movements contributing to the volume in the subject movement group.

The variable X_u is computed as the weighted volume-to-capacity ratio of all upstream movements contributing to the volume in the subject movement group. This ratio is computed as a weighted average with the volume-to-capacity ratio of each contributing upstream movement weighted by its discharge volume. For planning and design analyses, X_u can be approximated as the volume-to-capacity ratio of the contributing through movement at the upstream signalized intersection. The value of X_u used in Equation 18-3 cannot exceed 1.0.

Initial Queue

The initial queue represents the queue present at the start of the subject analysis period for the subject movement group. This queue is created when oversaturation is sustained for an extended time. The initial queue can be estimated by monitoring queue count continuously during each of the three consecutive cycles that occur just before the start of the analysis period. The smallest count observed during each cycle is recorded. The initial queue estimate equals the average of the three counts. The initial queue estimate should not include vehicles in the queue due to random, cycle-by-cycle fluctuations.

Base Saturation Flow Rate

The saturation flow rate represents the maximum rate of flow for a traffic lane, as measured at the stop line during the green indication. The base saturation flow rate represents the saturation flow rate for a traffic lane that is 12 ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. Typically, one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. It has units of passenger

cars per hour per lane (pc/h/ln). Chapter 31, Signalized Intersections: Supplemental, describes a field measurement technique for quantifying the local base saturation flow rate.

Lane Utilization Adjustment Factor

The lane utilization adjustment factor accounts for the unequal distribution of traffic among the lanes in those movement groups with more than one exclusive lane. This factor provides an adjustment to the base saturation flow rate to account for uneven use of the lanes. It is not used unless a movement group has more than one exclusive lane. It is calculated with Equation 18-4.

$$f_{LU} = \frac{v_g}{N_e \, v_{g1}}$$

Equation 18-4

where

f_{LU} = adjustment factor for lane utilization,

v_g = demand flow rate for movement group (veh/h),

v_{g1} = demand flow rate in the single exclusive lane with the highest flow rate of all exclusive lanes in movement group (veh/h/ln), and

N_e = number of exclusive lanes in movement group (ln).

Lane flow rates measured in the field can be used with Equation 18-4 to establish local default values of the lane utilization adjustment factor.

A lane utilization factor of 1.0 is used when a uniform traffic distribution can be assumed across all exclusive lanes in the movement group or when a movement group has only one lane. Values less than 1.0 apply when traffic is not uniformly distributed. As demand approaches capacity, the lane utilization factor is often closer to 1.0 because drivers have less opportunity to select their lane.

At some intersections, drivers may choose one through lane over another lane in anticipation of a turn at a downstream intersection. When this type of "prepositioning" occurs, a more accurate evaluation will be obtained when the actual flow rate for each approach lane is measured in the field and provided as an input to the methodology.

Pedestrian Flow Rate

The pedestrian flow rate is based on the count of pedestrians traveling in the crosswalk that is crossed by vehicles turning right from the subject approach during the analysis period. For example, the pedestrian flow rate for the westbound approach describes the pedestrian flow in the crosswalk on the north leg. A separate count is taken for each direction of travel in the crosswalk. Each count is divided by the analysis period duration to yield a directional hourly flow rate. These rates are then added to obtain the pedestrian flow rate.

Bicycle Flow Rate

The bicycle flow rate is based on the count of bicycles whose travel path is crossed by vehicles turning right from the subject approach during the analysis

period. These bicycles may travel on the shoulder or in a bike lane. Any bicycle traffic operating in the right lane with automobile traffic should not be included in this count. This interaction is not modeled by the methodology. The count is divided by the analysis period duration to yield an hourly flow rate.

On-Street Parking Maneuver Rate

The parking maneuver rate represents the count of *influential* parking maneuvers that occur on an intersection leg, as measured during the analysis period. An influential maneuver occurs directly adjacent to a movement group, within a zone that extends from the stop line to a point 250 ft upstream of it. A maneuver occurs when a vehicle enters or exits a parking stall. If more than 180 maneuvers/h exist, then a practical limit of 180 should be used. On a two-way leg, maneuvers are counted for just the right side of the leg. On a one-way leg, maneuvers are separately counted for each side of the leg. The count is divided by the analysis period duration to yield an hourly flow rate.

Local Bus Stopping Rate

The bus stopping rate represents the number of local buses that stop and block traffic flow in a movement group within 250 ft of the stop line (upstream or downstream), as measured during the analysis period. A *local bus* is a bus that stops to discharge or pick up passengers at a bus stop. The stop can be on the near side or the far side of the intersection. If more than 250 buses/h exist, then a practical limit of 250 should be used. The count is divided by the analysis period duration to yield an hourly flow rate.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 18-6. These data describe the geometric elements of the intersection that influence traffic operation.

Number of Lanes

The number of lanes represents the count of lanes provided for each intersection traffic movement. For a turn movement, this count represents the lanes reserved for the exclusive use of turning vehicles. Turn movement lanes include turn lanes that extend backward for the length of the segment and lanes in a turn bay. Lanes that are shared by two or more movements are included in the count of through lanes and are described as *shared lanes*. If no exclusive turn lanes are provided, then the turn movement is indicated to have 0 lanes.

Average Lane Width

The average lane width represents the average width of the lanes represented in a movement group. The minimum average lane width is 8 ft. Standard lane widths are 12 ft. Lane widths greater than 16 ft can be included; however, the analyst should consider whether the wide lane actually operates as two narrow lanes. The analysis should reflect the way in which the lane width is actually used or expected to be used.

Number of Receiving Lanes

The number of receiving lanes represents the count of lanes departing the intersection. This number should be separately determined for each left-turn and right-turn movement. Experience indicates that proper turning cannot be executed at some intersections because a receiving lane is frequently blocked by double-parked vehicles. For this reason, the number of receiving lanes should be determined from field observation when possible.

Turn Bay Length

Turn bay length represents the length of the bay for which the lanes have full width and in which queued vehicles can be stored. Bay length is measured parallel to the roadway centerline. If there are multiple lanes in the bay and they have different lengths, then the length entered should be an average value.

If a two-way left-turn lane is provided for left-turn vehicle storage and adjacent access points exist, then the bay length entered should represent the "effective" storage length available to the left-turn movement. The determination of effective length is based on consideration of the adjacent access points and the associated left-turning vehicles that store in the two-way left-turn lane.

Presence of On-Street Parking

This input indicates whether on-street parking is allowed along the curb line adjacent to a movement group and within 250 ft upstream of the stop line during the analysis period. On a two-way street, the presence of parking is noted for just the right side of the street. On a one-way street, the presence of on-street parking is separately noted for each side of the street.

Approach Grade

Approach grade defines the average grade along the approach, as measured from the stop line to a point 100 ft upstream of the stop line along a line parallel to the direction of travel. An uphill condition has a positive grade, and a downhill condition has a negative grade.

Signal Control Data

This subpart describes the signal control data listed in Exhibit 18-6 and Exhibit 18-7. They are specific to an actuated traffic signal controller that is operated in a pretimed, semiactuated, fully actuated, or coordinated-actuated manner.

Type of Signal Control

The methodology is based on the operation of a fully actuated controller. However, semiactuated, pretimed, and coordinated-actuated control can be achieved through proper specification of the controller inputs.

Semiactuated control is achieved by using the following settings for nonactuated phases:

- Maximum green is set to an appropriate value, and
- Maximum recall is invoked.

An equivalent pretimed control is achieved by using the following two settings for each signal phase:

- Maximum green is set to its desired pretimed green interval duration, and
- Maximum recall is invoked.

Settings used for coordinated-actuated control are described later in this subpart and are used in Chapter 17.

The automobile methodology is based on the latest controller functions defined in the National Transportation Communications for ITS Protocol Standard 1202. It is incumbent on the analyst to become familiar with these functions and adapt them, if needed, to the functionality of the controller that is used at the subject intersection. Chapter 31 provides additional information about traffic signal controller operation.

Phase Sequence

In a broad context, phase sequence describes the sequence of service provided to each traffic movement. This definition is narrowed here to limit phase sequence to a description of the order in which the left-turn movements are served, relative to the through movements. The sequence options addressed in the methodology include no left-turn phase, leading left-turn phase, lagging left-turn phase, and split phasing.

Left-Turn Operational Mode

The left-turn operational mode describes how the left-turn movement is served by the controller. It can be described as permitted, protected, or protected-permitted.

Dallas Left-Turn Phasing Option

This option allows the left-turn movements to operate in the protected-permitted mode without causing a "yellow trap" safety concern. It effectively ties the left turn's permitted period signal indication to the opposing through movement signal indication. This phasing option is also used with a flashing yellow arrow left-turn signal display.

Passage Time

Passage time is the maximum amount of time one vehicle actuation can extend the green interval while green is displayed. It is input for each actuated signal phase. It is also referred to as vehicle interval, extension interval, extension, or unit extension.

Passage time values are typically based on detection zone length, detection zone location (relative to the stop line), number of lanes served by the phase, and vehicle speed. Longer passage times are often used with shorter detection zones, greater distance between the zone and stop line, fewer lanes, and slower speeds.

The objective in determining the passage time value is to make it large enough to ensure that all queued vehicles are served but not so large that it extends for randomly arriving traffic. On high-speed approaches, this objective is broadened to include not making the passage time so long that the phase

frequently extends to its maximum setting (i.e., maxes out) so that safe phase termination is compromised.

Maximum Green

The maximum green setting defines the maximum amount of time that a green signal indication can be displayed in the presence of conflicting demand. Typical maximum green values for left-turn phases range from 15 to 30 s. Typical values for through phases serving the minor-street approach range from 20 to 40 s, and those for through phases serving the major-street approach range from 30 to 60 s.

For an operational analysis of pretimed operation, the maximum green setting for each phase should equal the desired green interval duration and the recall mode should be set to "maximum." These settings also apply to the major-street through-movement phases for semiactuated operation.

For an analysis of coordinated-actuated operation, the maximum green is disabled through the inhibit mode and the phase splits are used to determine the maximum length of the actuated phases.

Minimum Green

The minimum green setting represents the least amount of time a green signal indication is displayed when a signal phase is activated. Its duration is based on consideration of driver reaction time, queue size, and driver expectancy. Minimum green typically ranges from 4 to 15 s, with shorter values in this range used for phases serving turn movements and lower-volume through movements. For intersections without pedestrian push buttons, the minimum green setting may also need to be long enough to allow time for pedestrians to react to the signal indication and cross the street.

Yellow Change and Red Clearance

The yellow change and the red clearance settings are input for each signal phase. The yellow change interval is intended to alert a driver to the impending presentation of a red indication. It ranges from 3 to 6 s, with longer values in this range used with phases serving high-speed movements. The red clearance interval can be used to allow a brief time to elapse after the yellow indication, during which the signal heads associated with the ending phase and all conflicting phases display a red indication. If used, the red clearance interval is typically 1 or 2 s.

Walk

The walk interval is intended to give pedestrians adequate time to perceive the WALK indication and depart the curb before the pedestrian clear interval begins.

For an actuated or a noncoordinated phase, the walk interval is typically set at the minimum value needed for pedestrian perception and curb departure. Many agencies consider this value to be 7 s; however, some agencies use as little as 4 s. Longer walk durations should be considered in school zones and areas with large numbers of elderly pedestrians. In the methodology, it is assumed that

the rest-in-walk mode is not enabled for actuated phases and noncoordinated phases.

For a pretimed phase, the walk interval is often set at a value equal to the green interval duration needed for vehicle service less the pedestrian clear setting (provided that the resulting interval exceeds the minimum time needed for pedestrian perception and curb departure).

For a coordinated phase, the controller is sometimes set to use a coordination mode that extends the walk interval for most of the green interval duration. This functionality is not explicitly modeled in the automobile methodology, but it can be approximated by setting the walk interval to a value equal to the phase split minus the sum of the pedestrian clear, yellow change, and red clearance intervals.

If the walk and pedestrian clear settings are provided for a phase, then it is assumed that a pedestrian signal head is also provided. If these settings are not used, then it is assumed that any pedestrian accommodation needed is provided in the minimum green setting.

Pedestrian Clear

The pedestrian clear interval (also referred to as the pedestrian change interval) is intended to provide time for pedestrians who depart the curb during the WALK indication to reach the opposite curb (or the median). Some agencies set the pedestrian clear equal to the "crossing time," where crossing time equals the curb-to-curb crossing distance divided by the pedestrian walking speed of 3.5 ft/s. Other agencies set the pedestrian clear equal to the crossing time less the vehicle change period (i.e., the combined yellow change and red clearance intervals). This choice depends on agency policy and practice. A flashing DON'T WALK indication is displayed during this interval.

Phase Recall

If used, recall causes the controller to place a call for a specified phase each time the controller is servicing a conflicting phase. It is input for each signal phase. Three types of recalls are modeled in the automobile methodology: minimum recall, maximum recall, and pedestrian recall.

Invoking minimum recall causes the controller to place a continuous call for vehicle service on the phase and then service the phase until its minimum green interval times out. The phase can be extended if actuations are received.

Invoking maximum recall causes the controller to place a continuous call for vehicle service on the phase. It results in presentation of the green indication for its maximum duration every cycle. Using maximum recall on all phases yields an equivalent pretimed operation.

Invoking pedestrian recall causes the controller to place a continuous call for pedestrian service on the phase and then service the phase for at least an amount of time equal to its walk and pedestrian clear intervals (longer if vehicle detections are received). Pedestrian recall is used for phases that have a high probability of pedestrian demand every cycle and no pedestrian detection.

Dual Entry

The entry mode is used in dual-ring operation to specify whether a phase is to be activated (green) even though it has not received a call for service. Two entry modes are possible: dual entry and single entry. This mode is input for each actuated signal phase.

A phase operating in dual entry is available to be called by the controller, even if no actuations have been received for this phase. A phase operating in single entry will be called only if actuations have been received.

During the timing of a cycle, a point is reached where the next phase (or phases) to be timed is on the other side of a barrier. At this point, the controller will check the phases in each ring and determine which phase to activate. If a call does not exist in a ring, the controller will activate a phase designated as dual entry in that ring. If two phases are designated as dual entry in the ring, then the first phase to occur in the phase sequence is activated.

Simultaneous Gap-Out

The simultaneous gap-out mode affects the way actuated phases are terminated before the barrier can be crossed to serve a conflicting call. This mode can be enabled or disabled. It is a phase-specific setting; however, it is typically set the same for all phases that serve the same street. This mode is input for each actuated signal phase.

Simultaneous gap-out dictates controller operation when a barrier must be crossed to serve the next call and one phase is active in each ring. If simultaneous gap-out is enabled, it requires that both phases reach a point of being committed to terminate (via gap out, max out, or force-off) at the same time. If one phase is able to terminate because it has gapped out, but the other phase is not able to terminate, then the gapped-out phase will reset its extension timer and restart the process of timing down to gap-out.

If the simultaneous gap-out feature is disabled, then each phase can reach a point of termination independently. In this situation, the first phase to commit to termination maintains its active status while waiting for the other phase to commit to termination. Regardless of which mode is in effect, the barrier is not crossed until both phases are committed to terminate.

Cycle Length (Coordinated-Actuated Operation)

Cycle length is the time elapsed between the endings of two sequential presentations of a coordinated phase green interval.

Phase Splits (Coordinated-Actuated Operation)

Each noncoordinated phase is provided a “split” time. This time represents the sum of the green, yellow change, and red clearance intervals for the phase. The rationale for determining the green interval duration varies among agencies; however, it is often related to the “optimum” pretimed green interval duration. Chapter 31 describes a procedure for determining pretimed phase duration.

Offset and Offset Reference Point (Coordinated-Actuated Operation)

The reference phase is specified to be one of the two coordinated phases (i.e., Phase 2 or 6). The offset entered in the controller represents the time that the reference phase begins (or ends) relative to the system master time zero. The offset must be specified as being referenced to the beginning, or the end, of the green interval of the reference phase. The offset reference point is typically the same at all intersections in a given signal system.

Force Mode (Coordinated-Actuated Operation)

This mode is a controller-specific setting. It is set to "fixed" or "floating." The controller calculates the phase force-off point for each noncoordinated phase on the basis of the force mode and the phase splits. When set to the fixed mode, each noncoordinated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. This operation allows unused split time to revert to the following phase. When set to the floating mode, each noncoordinated phase has its force-off point set at the split time after the phase first becomes active. This operation allows unused split time to revert to the coordinated phase (referred to as an "early return to green").

Other Data

This subpart describes the data listed in Exhibit 18-6 that are categorized as "other" data.

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. It ranges from 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should interpret the results from an analysis period of 1 h or more with caution because the adverse impact of short peaks in traffic demand may not be detected. Also, if the analysis period is other than 15 min, then the peak hour factor should not be used.

The methodology was developed to evaluate conditions in which queue spillback does not affect the performance of the subject intersection or any upstream intersection during the analysis period. If spillback affects intersection performance, the analyst should consider use of an alternative analysis tool that is able to model the effect of spillback conditions.

Operational Analysis. A 15-min analysis period should be used for operational analyses. This duration will accurately capture the adverse effects of demand peaks. Any 15-min period of interest can be evaluated with the methodology; however, a complete evaluation should always include an analysis of conditions during the 15-min period that experiences the highest traffic demand during a 24-h period.

If traffic demand exceeds capacity for a given 15-min analysis period, then a multiple-period analysis should be conducted. This type of analysis consists of an evaluation of several consecutive 15-min time periods. The periods analyzed would include an initial analysis period that has no initial queue, one or more

periods in which demand exceeds capacity, and a final analysis period that has no residual queue.

When a multiple-period analysis is used, intersection performance measures are computed for each analysis period. Averaging performance measures across multiple analysis periods is not encouraged because it may obscure extreme values.

Planning Analysis. A 15-min analysis period is used for most planning analyses. However, hourly traffic demands are normally produced through the planning process. Thus, when 15-min forecast demands are not available for a 15-min analysis period, a peak hour factor must be used to estimate the 15-min demands for the analysis period. A 1-h analysis period can be used, if appropriate. Regardless of analysis period duration, a single-period analysis is typical for planning applications.

Speed Limit

Average running speed is used in the methodology to evaluate lane group performance. It is correlated with speed limit when speed limit reflects the environmental and geometric factors that influence driver speed choice. As such, speed limit represents a single input variable that can be used as a convenient way to estimate running speed while limiting the need for numerous environmental and geometric input data.

The convenience of using speed limit as an input variable comes with a caution—the analyst must not infer a cause-and-effect relationship between the input speed limit and the estimated running speed. More specifically, the computed change in performance resulting from a change in the input speed limit is not likely to be indicative of performance changes that will actually be realized. Research indicates that a change in speed limit has a proportionally smaller effect on the actual average speed (*24*).

The methodology is based on the assumption that the posted speed limit is (*a*) consistent with that found on other streets in the vicinity of the subject intersection and (*b*) consistent with agency policy regarding specification of speed limits. If it is known that the posted speed limit does not satisfy these assumptions, then the speed limit value that is input to the methodology should be adjusted so that it is consistent with the assumptions.

Stop-Line Detector Length and Detection Mode

The stop-line detector length represents the length of the detection zone used to extend the green indication. This detection zone is typically located near the stop line and may have a length of 40 ft or more. However, it can be located some distance upstream of the stop line and may be as short as 6 ft. The latter configuration typically requires a long minimum green or use of the controller's variable initial setting.

If a video-image vehicle detection system is used to provide stop-line detection, then the length that is input should reflect the physical length of roadway that is monitored by the video detection zone plus a length of 5 to 10 ft

to account for the projection of the vehicle image into the plane of the pavement (with larger values in this range used for wider intersections).

Detection mode influences the duration of the actuation submitted to the controller by the detection unit. One of two modes can be used: presence or pulse. Presence mode is typically the default mode. It tends to provide more reliable intersection operation than pulse mode.

In the presence mode, the actuation starts with the vehicle arriving in the detection zone and ends with the vehicle leaving the detection zone. Thus, the time duration of the actuation depends on vehicle length, detection zone length, and vehicle speed.

The presence mode is typically used with long detection zones located at the stop line. The combination typically results in the need for a small passage time value. This characteristic is desirable because it tends to result in efficient queue service.

In the pulse mode, the actuation starts and ends with the vehicle arriving at the detector (actually, the actuation is a short "on" pulse of 0.10 to 0.15 s). This mode is not used as often as presence mode for intersection control.

Area Type

The area type input is used to indicate whether the intersection is in a central business district (CBD) type of environment. An intersection is considered to be in a CBD, or a similar type of area, when its characteristics include narrow street rights-of-way, frequent parking maneuvers, vehicle blockages, taxi and bus activity, small-radius turns, limited use of exclusive turn lanes, high pedestrian activity, dense population, and midblock curb cuts. The average saturation headway at intersections in areas with these characteristics is significantly longer than that found at intersections in areas that are less constrained and less visually intense.

Nonautomobile Modes

This part describes the input data needed for the pedestrian and bicycle methodologies. The data are listed in Exhibit 18-9 and are identified as "input data elements."

Exhibit 18-9 categorizes each input data element by travel mode methodology. The association between a data element and its travel mode is indicated by the provision of text in the corresponding cell of Exhibit 18-9. When text is provided in a cell, it indicates whether the data are needed for a traffic movement, signal phase, intersection approach, intersection leg, or intersection as a whole. A blank cell indicates that the data element is not an input for the corresponding travel mode.

The data elements listed in Exhibit 18-9 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during presentation of the methodology.

Exhibit 18-9
Input Data Requirements: Nonautomobile Modes

Data Category	Input Data Element	Pedestrian Mode [a]	Bicycle Mode [a]
Traffic characteristics	Demand flow rate of motorized vehicles	Movement	Approach
	Right-turn-on-red flow rate	Approach	
	Permitted left-turn flow rate	Movement	
	Midsegment 85th percentile speed	Approach	
	Pedestrian flow rate	Movement	
	Bicycle flow rate		Approach
	Proportion of on-street parking occupied		Approach
Geometric design	Street width		Approach
	Number of lanes	Leg	Approach
	Number of right-turn islands	Leg	
	Width of outside through lane		Approach
	Width of bicycle lane		Approach
	Width of paved outside shoulder (or parking lane)		Approach
	Total walkway width	Approach	
	Crosswalk width	Leg	
	Crosswalk length	Leg	
	Corner radius	Approach	
Signal control	Walk	Phase	
	Pedestrian clear	Phase	
	Rest in walk	Phase	
	Cycle length	Intersection	Intersection
	Yellow change	Phase	Phase
	Red clearance	Phase	Phase
	Duration of phase serving pedestrians and bicycles	Phase	Phase
	Pedestrian signal head presence	Phase	
Other	Analysis period duration [b]	Intersection	Intersection

Notes: [a] Movement = one value for each left-turn, through, and right-turn movement.
Approach = one value for the intersection approach.
Leg = one value for the intersection leg (approach plus departure sides).
Intersection = one value for the intersection.
Phase = one value or condition for each signal phase.
[b] Analysis period duration is as defined for Exhibit 18-6.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 18-9. These data describe the traffic streams traveling through the intersection during the study period. The demand flow rate of motorized vehicles, RTOR flow rate, and bicycle flow rate were defined in the previous subsection for the automobile mode.

Permitted Left-Turn Flow Rate

The permitted left-turn flow rate is defined as the count of vehicles that turn left permissively, divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. A permitted left-turn movement can occur with either the permitted or the protected-permitted left-turn mode. For left-turn movements served by the permitted mode, the permitted left-turn flow rate is equal to the left-turn demand flow rate.

For left-turn movements served by the protected-permitted mode, the permitted left-turn flow rate should be measured in the field because its value is

influenced by many factors. Section 3, Applications, describes a procedure that can be used to estimate a default flow rate if the analysis involves future conditions or if the permitted left-turn flow rate is not known from field data.

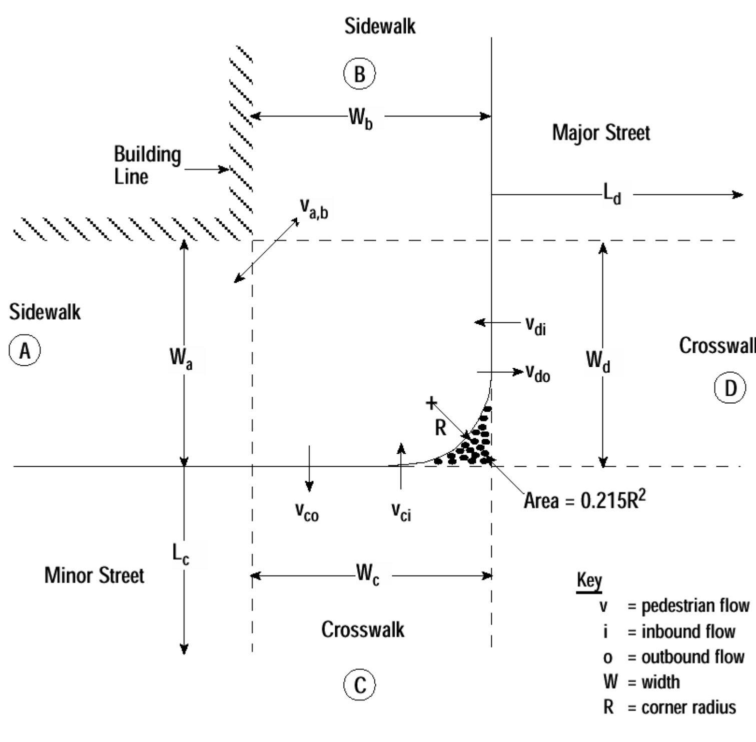
Midsegment 85th Percentile Speed

The 85th percentile speed represents the speed of the vehicle whose speed is exceeded by only 15% of the population of vehicles. The speed of interest is that of vehicles traveling along the street approaching the subject intersection. It is measured at a location sufficiently distant from the intersection that speed is not influenced by intersection operation. This speed is likely to be influenced by traffic conditions, so it should reflect the conditions present during the analysis period.

Pedestrian Flow Rate

The pedestrian flow rate represents the count of pedestrians traveling through each corner of the intersection divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. This flow rate is provided for each of five movements at each intersection corner. These five movements (i.e., v_{ci}, v_{co}, v_{di}, v_{do}, and $v_{a,b}$) are shown in Exhibit 18-10 as they occur at one intersection corner.

Exhibit 18-10
Intersection Corner Geometry and Pedestrian Movements



Proportion of On-Street Parking Occupied

This variable represents the proportion of the intersection's right-side curb line that has parked vehicles present during the analysis period. It is based on a zone that extends from a point 250 ft upstream of the intersection to the intersection, and a second zone that extends from the intersection to a point

250 ft downstream of the intersection. If parking is not allowed in these two zones, then this proportion equals 0.0.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 18-9. These data describe the geometric elements that influence intersection performance from a pedestrian or bicyclist perspective. The number-of-lanes variable was defined in the previous subsection for the automobile mode.

Street Width

The street width represents the width of the cross street as measured along the outside through vehicle lane on the subject approach between the extended curb line limits of the cross street. It is measured for each intersection approach.

Width of Through Lane, Width of Bicycle Lane, and Width of Shoulder

Several individual elements of the cross section are described in this subpart. These elements include the width of the outside through vehicle lane, the bicycle lane adjacent to the outside lane, and the paved outside shoulder.

The width of each of these elements is mutually exclusive (i.e., not overlapped). The outside lane width does not include the width of the gutter.

Total Walkway Width, Crosswalk Width and Length, and Corner Radius

These geometric design data describe the pedestrian accommodations on each corner of the intersection. These data are shown in Exhibit 18-10. The total walkway width (i.e., W_a and W_b) is measured from the outside edge of the road pavement (or face of curb, if present) to the far edge of the sidewalk (as sometimes delineated by building face, fence, or landscaping).

The crosswalk width (i.e., W_c and W_d) represents an effective width. Unless there is a known width constraint, the crosswalk's effective width should be the same as its physical width. A width constraint may be found when vehicles are observed to encroach regularly into the crosswalk area or when an obstruction in the median (e.g., a signal pole or reduced-width cut in the median curb) narrows the walking space.

The crosswalk length (i.e., L_c and L_d) is measured from outside edge to outside edge of road pavement (or curb to curb, if present) along the marked pedestrian travel path.

Signal Control Data

This subpart describes the data in Exhibit 18-9 that are identified as "signal control." The walk, pedestrian clear, yellow change, and red clearance settings were defined in the previous subsection for the automobile mode.

Rest in Walk

A phase with the rest-in-walk mode enabled will dwell in walk as long as there are no conflicting calls. When a conflicting call is received, the pedestrian clear interval will time to its setting value before ending the phase. This mode can be enabled for any actuated phase. Signals that operate with coordinated-

actuated operation may be set to use a coordination mode that enables the rest-in-walk mode. Typically, the rest-in-walk mode is not enabled. In this case, the walk and pedestrian clear intervals time to their respective setting values, and then the pedestrian signal indication dwells in a steady DON'T WALK indication until a conflicting call is received.

Cycle Length

Cycle length is predetermined for pretimed or coordinated-actuated control. Chapter 31 provides a procedure for estimating a reasonable cycle length for these two types of control when cycle length is unknown.

For semiactuated and fully actuated control, an average cycle length must be provided as input to use the pedestrian or bicycle methodologies. This length can be estimated by using the automobile methodology.

Pedestrian Signal Head Presence

The presence of a pedestrian signal head influences pedestrian crossing behavior. If a pedestrian signal head is provided, then pedestrians are assumed to use the crosswalk during the WALK and flashing DON'T WALK indications. If no pedestrian signal heads are provided, then pedestrians will cross during the green indication provided to vehicular traffic.

Duration of Phase Serving Pedestrians and Bicycles

The duration of each phase that serves a pedestrian or bicycle movement is a required input. This phase is typically the phase that serves the through movement that is adjacent to the sidewalk and for which the pedestrian, bicycle, and through vehicle travel paths are parallel. For example, Phases 2, 4, 6, and 8 are the phases serving the pedestrian and bicycle movements in Exhibit 18-3.

SCOPE OF THE METHODOLOGY

Three methodologies are presented in this chapter, one for each of the automobile, pedestrian, and bicycle modes. This subsection identifies the conditions for which each methodology applies.

- *Signalized intersections.* All methodologies can be used to evaluate intersection performance from the perspective of the corresponding travel mode. The automobile methodology is developed to replicate fully actuated controller operation. However, specific inputs to the methodology can be used to facilitate evaluation of coordinated-actuated, semiactuated, or pretimed control.
- *Steady flow conditions.* The three methodologies are based on the analysis of steady traffic conditions and, as such, are not well suited to the evaluation of unsteady conditions (e.g., congestion, queue spillback, signal preemption).
- *Target road users.* Collectively, the three methodologies were developed to estimate the LOS perceived by automobile drivers, pedestrians, and bicyclists. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers,

automobile passengers, delivery truck drivers, recreational vehicle drivers). However, it is likely that the perceptions of these other road users are reasonably well represented by the road users for whom the methodologies were developed.

- *Target travel modes.* The automobile methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams where the automobile represents the largest percentage of all vehicles. The pedestrian and bicycle methodologies address travel by walking and bicycle, respectively. The methodologies are not designed to evaluate the performance of other types of vehicles (e.g., golf carts, motorized bicycles).
- *Influences in the right-of-way.* A road user's perception of quality of service is influenced by many factors inside and outside the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside the right-of- way (e.g., buildings, parking lots, scenery, landscaped yards) that might influence a traveler's perspective. This approach was followed because factors outside the right-of-way are not under the direct control of the agency operating the street.
- *"Typical pedestrian" focus for pedestrian methodology.* The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. As such, the performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to Americans with Disabilities Act (ADA) requirements. For this reason, they should not be considered as a substitute for an ADA compliance assessment of a pedestrian facility.

LIMITATIONS OF THE METHODOLOGY

In general, the methodologies described in this chapter can be used to evaluate the performance of most traffic streams traveling through an intersection. However, the methodologies do not address all traffic conditions or intersection configurations. The inability to replicate the influence of a condition or configuration in the methodology represents a limitation. This subsection identifies the known limitations of the methodologies described in this chapter. If one or more of these limitations is believed to have an important influence on the performance of a specific intersection, then the analyst should consider the use of alternative methods or tools.

Automobile Mode

The automobile methodology does not explicitly account for the effect of the following conditions on intersection operation:

- Turn bay overflow;
- Multiple advance detectors in the same lane;
- Demand starvation due to a closely spaced upstream intersection;

- Queue spillback into the subject intersection from a downstream intersection;
- Queue spillback from the subject intersection into an upstream intersection;
- Premature phase termination due to short detection length, passage time, or both;
- RTOR volume prediction or resulting right-turn delay;
- Turn movements served by more than two exclusive lanes;
- A right-turn movement that is not under signal control;
- Through lane (or lanes) added just upstream of the intersection or dropped just downstream of the intersection; and
- Storage of shared-lane left-turning vehicles within the intersection to permit bypass by through vehicles in the same lane.

In addition to the above conditions, the methodology does not directly account for the following controller functions:

- Rest-in-walk mode for actuated and noncoordinated phases,
- Preemption or priority modes,
- Phase overlap, and
- Gap reduction or variable initial settings for actuated phases.

Nonautomobile Modes

This part identifies the limitations of the pedestrian and bicycle methodologies. These methodologies are not able to model the conditions offered in the following list:

- Presence of grades in excess of 2%, and
- Presence of railroad crossings.

In addition, the pedestrian methodology does not model the following conditions:

- Unpaved sidewalk, and
- Free (i.e., uncontrolled) channelized right turn with multiple lanes or high-speed operation.

2. METHODOLOGY

OVERVIEW

This section describes three methodologies for evaluating the performance of a signalized intersection. Each methodology addresses one possible travel mode through the intersection. Analysts should choose the combination of methodologies that are appropriate for their analysis needs.

A complete evaluation of intersection operation includes the separate examination of performance for all relevant travel modes. The performance measures associated with each mode are assessed independently of one another. They are not mathematically combined into a single indicator of intersection performance. This approach ensures that all performance impacts are considered on a mode-by-mode basis.

The focus of each methodology in this chapter is the signalized intersection. Chapter 17, Urban Street Segments, provides a methodology for quantifying the performance of an urban street segment. The methodology described in Chapter 16, Urban Street Facilities, can be used to combine the performance measures (for a specified travel mode) on successive segments into an overall measure of facility performance for that mode.

AUTOMOBILE MODE

This subsection provides an overview of the methodology for evaluating signalized intersection performance from the motorist perspective. The methodology is computationally intense and requires software to implement. The intensity stems partly from the need to model traffic-actuated signal operation. Default values are provided in Section 3, Applications, to support planning analyses for which the required input data are not available.

A quick estimation method for evaluating intersection performance at a planning level of analysis is provided in Chapter 31, Signalized Intersections: Supplemental. This method is not computationally intense and can be applied by using hand calculations.

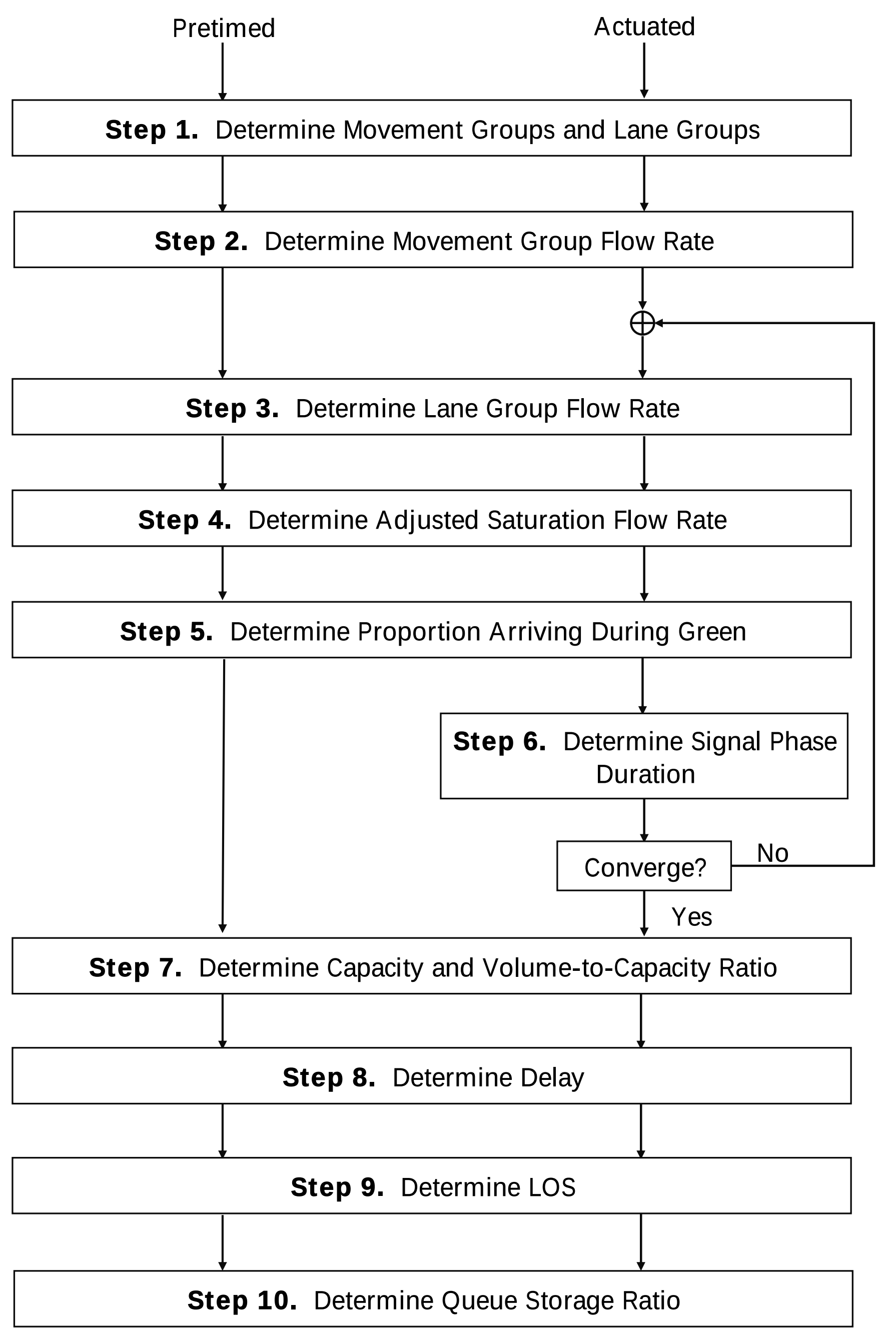
Because of the intensity of the computations, the objective of this subsection is to introduce the analyst to the calculation process and discuss the key analytic procedures. This objective is achieved by focusing the discussion on lane groups that serve one traffic movement with pretimed control and for which there are no permitted or protected-permitted left-turn movements. Details on evaluation of actuated control, shared-lane lane groups, and intersections with permitted or protected-permitted left-turn operation are provided in Chapter 31.

The computational engine developed by the Transportation Research Board Committee on Highway Capacity and Quality of Service represents the most detailed description of this methodology. Additional information about this engine is provided in Chapter 31.

Framework

Exhibit 18-11 illustrates the calculation framework of the automobile methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. These calculations are described more fully in the remainder of this subsection.

Exhibit 18-11
Automobile Methodology for Signalized Intersections



Step 1: Determine Movement Groups and Lane Groups

The methodology for signalized intersections uses the concept of *movement groups* and *lane groups* to describe and evaluate intersection operation. These two group designations are very similar in meaning. In fact, their differences emerge only when a shared lane is present on an approach with two or more lanes. Each designation is defined in the following paragraphs. The movement-group designation is a useful construct for specifying input data. In contrast, the lane-group designation is a useful construct for describing the calculations associated with the methodology.

The following rules are used to determine movement groups for an intersection approach:

- A turn movement that is served by one or more exclusive lanes and no shared lanes should be designated as a movement group.
- Any lanes not assigned to a group by the previous rule should be combined into one movement group.

These rules result in the designation of one to three movement groups for each approach.

The concept of lane groups is useful when a shared lane is present on an approach that has two or more lanes. Several procedures in the methodology require some indication of whether the shared lane serves a mix of vehicles or functions as an exclusive turn lane. This issue cannot be resolved until the proportion of turns in the shared lane has been computed. If the computed proportion of turns in the shared lane equals 1.0 (i.e., 100%), the shared lane is considered to operate as an exclusive turn lane.

The following rules are used to determine lane groups for an intersection approach:

- An exclusive left-turn lane or lanes should be designated as a separate lane group. The same is true of an exclusive right-turn lane.
- Any shared lane should be designated as a separate lane group.
- Any lanes that are not exclusive turn lanes or shared lanes should be combined into one lane group.

These rules result in the designation of one or more of the following lane group possibilities for an intersection approach:

- Exclusive left-turn lane (or lanes),
- Exclusive through lane (or lanes),
- Exclusive right-turn lane (or lanes),
- Shared left-turn and through lane,
- Shared left-turn and right-turn lane,
- Shared right-turn and through lane, and
- Shared left-turn, through, and right-turn lane.

The methodology can be applied to any logical combination of these lane groups. Exhibit 18-12 shows some common movement groups and lane groups.

Exhibit 18-12
Typical Lane Groups for Analysis

Number of Lanes	Movements by Lanes	Movement Groups (MG)	Lane Groups (LG)
1	Left, thru., & right:	MG 1:	LG 1:
2	Exclusive left: Thru. & right:	MG 1: MG 2:	LG 1: LG 2:
2	Left & thru.: Thru. & right:	MG 1:	LG 1: LG 2:
3	Exclusive left: Exclusive left: Through: Through: Thru. & right:	MG 1: MG 2:	LG 1: LG 2: LG 3:

Step 2: Determine Movement Group Flow Rate

The flow rate for each movement group is determined in this step. If a turn movement is served by one or more exclusive lanes and no shared lanes, then that movement's flow rate is assigned to a movement group. Any of the approach flow that is yet to be assigned to a movement group (following application of the guidance in the previous sentence) is assigned to one movement group.

The RTOR flow rate is subtracted from the right-turn flow rate, regardless of whether the right turn occurs from a shared or an exclusive lane. At an existing intersection, the number of RTORs should be determined by field observation.

Step 3: Determine Lane Group Flow Rate

The lane group flow rate is determined in this step. If there are no shared lanes on the intersection approach or the approach has only one lane, there is a one-to-one correspondence between lane groups and movement groups. In this situation, the lane group flow rate equals the movement group flow rate.

If there are one or more shared lanes on the approach and two or more lanes, then the lane group flow rate is computed by the procedure described in Chapter 31. This procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection, where the lane volume-to-saturation flow ratio is used to estimate relative differences in this time among lanes. This assumption may not always hold for situations in which drivers choose a lane on the subject approach so that they are prepositioned for a turn at a downstream intersection. In this situation, the analyst needs to provide as input the demand flow rate for each lane on the approach and aggregate them as appropriate to define the lane group flow rate.

Step 4: Determine Adjusted Saturation Flow Rate

The adjusted saturation flow rate for each lane of each lane group is computed in this step. The base saturation flow rate provided as an input variable is used in this computation.

The computed saturation flow rate is referred to as the "adjusted" saturation flow rate because it reflects the application of various factors that adjust the base saturation flow rate to the specific conditions present on the subject intersection approach.

The procedure described in this step applies to lane groups that consist of an exclusive lane (or lanes) operating in a pretimed protected mode and without pedestrian or bicycle interaction. When these conditions do not hold, the supplemental procedures described in Chapter 31 should be combined with those in this step to compute the adjusted saturation flow rate.

Equation 18-5 is used to compute the adjusted saturation flow rate per lane for the subject lane group:

$$s = s_o\, f_w\, f_{HV}\, f_g\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{RT}\, f_{Lpb}\, f_{Rpb}$$

Equation 18-5

where

s = adjusted saturation flow rate (veh/h/ln),

s_o = base saturation flow rate (pc/h/ln),

f_w = adjustment factor for lane width,

f_{HV} = adjustment factor for heavy vehicles in traffic stream,

f_g = adjustment factor for approach grade,

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group,

f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area,

f_a = adjustment factor for area type,

f_{LU} = adjustment factor for lane utilization,

f_{LT} = adjustment factor for left-turn vehicle presence in a lane group,

f_{RT} = adjustment factor for right-turn vehicle presence in a lane group,

f_{Lpb} = pedestrian adjustment factor for left-turn groups, and

f_{Rpb} = pedestrian–bicycle adjustment factor for right-turn groups.

The adjustment factors in the list above are described in the following subparts.

Base Saturation Flow Rate

Computations begin with selection of a base saturation flow rate. This base rate represents the expected average flow rate for a through-traffic lane having geometric and traffic conditions that correspond to a value of 1.0 for each adjustment factor. Typically, one base rate is selected to represent all signalized

intersections in the jurisdiction (or area) within which the subject intersection is located. Default values for this rate are provided in Section 3, Applications.

Adjustment for Lane Width

The lane width adjustment factor f_w accounts for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes. Values of this factor are listed in Exhibit 18-13.

Exhibit 18-13
Lane Width Adjustment Factor

Average Lane Width (ft)	Adjustment Factor (f_w)
<10.0[a]	0.96
≥10.0–12.9	1.00
>12.9	1.04

Note: [a] Factors apply to average lane widths of 8.0 ft or more.

Standard lanes are 12 ft wide. The lane width factor may be used with caution for lane widths greater than 16 ft, or an analysis with two narrow lanes may be conducted. Use of two narrow lanes will always result in a higher saturation flow rate than a single wide lane, but, in either case, the analysis should reflect the way the width is actually used or expected to be used. In no case should this factor be used to estimate the saturation flow rate of a lane group with an average lane width that is less than 8.0 ft.

Adjustment for Heavy Vehicles

The heavy-vehicle adjustment factor f_{HV} accounts for the additional space occupied by heavy vehicles and for the difference in their operating capabilities, compared with passenger cars. This factor does not address local buses that stop in the intersection area. Values of this factor are computed with Equation 18-6.

Equation 18-6

$$f_{HV} = \frac{100}{100 + P_{HV}(E_T - 1)}$$

where

P_{HV} = percent heavy vehicles in the corresponding movement group (%), and

E_T = equivalent number of through cars for each heavy vehicle = 2.0.

Adjustment for Grade

The grade adjustment factor f_g accounts for the effects of approach grade on vehicle performance. Values of this factor are computed with Equation 18-7.

Equation 18-7

$$f_g = 1 - \frac{P_g}{200}$$

where P_g is the approach grade for the corresponding movement group (%).

This factor applies to grades ranging from –6.0% to +10.0%. An uphill grade has a positive value and a downhill grade has a negative value.

Adjustment for Parking

The parking adjustment factor f_p accounts for the frictional effect of a parking lane on flow in the lane group adjacent to the parking lane. It also accounts for the occasional blocking of an adjacent lane by vehicles moving into and out of

parking spaces. If no parking is present, then this factor has a value of 1.00. If parking is present, then the value of this factor is computed with Equation 18-8.

$$f_p = \frac{N - 0.1 - \dfrac{18N_m}{3{,}600}}{N} \geq 0.050$$

Equation 18-8

where

N_m = parking maneuver rate adjacent to lane group (maneuvers/h), and

N = number of lanes in lane group (ln).

The parking maneuver rate corresponds to parking areas directly adjacent to the lane group and within 250 ft upstream of the stop line. A practical upper limit of 180 maneuvers/h should be maintained with Equation 18-8. A minimum value of f_p from this equation is 0.050. Each maneuver (either in or out) is assumed to block traffic in the lane next to the parking maneuver for an average of 18 s.

The factor applies only to the lane group that is adjacent to the parking. On a one-way street with a single-lane lane group, the number of maneuvers used is the total for both sides of the lane group. On a one-way street with two or more lane groups, the factor is calculated separately for each lane group and is based on the number of maneuvers adjacent to the group. Parking conditions with zero maneuvers have an impact different from that of a no-parking situation.

Adjustment for Bus Blockage

The bus-blockage adjustment factor f_{bb} accounts for the impact of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 250 ft of the stop line (upstream or downstream). Values of this factor are computed with Equation 18-9.

$$f_{bb} = \frac{N - \dfrac{14.4N_b}{3{,}600}}{N} \geq 0.050$$

Equation 18-9

where N is the number of lanes in lane group (ln) and N_b is the bus stopping rate on the subject approach (buses/h).

This factor should be used only when stopping buses block traffic flow in the subject lane group. A practical upper limit of 250 buses/h should be maintained with Equation 18-9. A minimum value of f_{bb} from this equation is 0.050. The factor used here assumes an average blockage time of 14.4 s during a green indication.

Adjustment for Area Type

The area type adjustment factor f_a accounts for the inefficiency of intersections in CBDs relative to those in other locations. When used, it has a value of 0.90.

Use of this factor should be determined on a case-by-case basis. This factor is not limited to designated CBD areas, nor does it need to be used for all CBD areas. Instead, it should be used in areas where the geometric design and the

traffic or pedestrian flows, or both, are such that the vehicle headways are significantly increased.

Adjustment for Lane Utilization

The input lane utilization adjustment factor is used to estimate saturation flow rate for a lane group with more than one exclusive lane. If the lane group has one shared lane or one exclusive lane, then this factor is 1.0.

Adjustment for Right Turns

The right-turn adjustment factor f_{RT} is intended primarily to reflect the effect of right-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 18-10.

Equation 18-10

$$f_{RT} = \frac{1}{E_R}$$

where E_R is the equivalent number of through cars for a protected right-turning vehicle (= 1.18).

If the right-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians and bicycles on right-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Left Turns

The left-turn adjustment factor f_{LT} is intended primarily to reflect the effect of left-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 18-11.

Equation 18-11

$$f_{LT} = \frac{1}{E_L}$$

where E_L is the equivalent number of through cars for a protected left-turning vehicle (= 1.05).

If the left-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians on left-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Pedestrians and Bicycles

The procedure to determine the left-turn pedestrian–bicycle adjustment factor f_{Lpb} and the right-turn pedestrian–bicycle adjustment factor f_{Rpb} is based on the concept of conflict zone occupancy, which accounts for the conflict between turning vehicles, pedestrians, and bicycles. Relevant conflict zone occupancy takes into account whether the opposing vehicle flow is also in conflict with the left-turn movement. The proportion of green time in which the conflict zone is occupied is determined as a function of the relevant occupancy and the number

of receiving lanes for the turning vehicles. A procedure for computing these factors is provided in Chapter 31.

Step 5: Determine Proportion Arriving During Green

Control delay and queue size at a signalized intersection are highly dependent on the proportion of vehicles that arrive during the green and red signal indications. Delay and queue size are smaller when a larger proportion of vehicles arrive during the green indication. Equation 18-12 is used to compute this proportion for each lane group.

$$P = R_p (g / C)$$

Equation 18-12

All variables are as previously defined. This equation requires knowledge of the effective green time g and cycle length C. These values are known for pretimed operation. If the intersection is not pretimed, then the average phase time and cycle length must be calculated by the procedures described in the next step.

The procedure in Chapter 17 can be used to estimate the arrival flow profile for an intersection approach when this approach is evaluated as part of an urban street segment. The procedure uses the profile to compute the proportion of arrivals during the green indication.

Step 6: Determine Signal Phase Duration

The duration of a signal phase depends on the type of control used at the subject intersection. If the intersection has pretimed control, then the phase duration is an input and this step is skipped. If the phase duration is unknown, then the pretimed phase duration procedure in Section 2 of Chapter 31 can be used to estimate the pretimed phase duration.

If the intersection has actuated control, then the actuated phase duration procedure in Section 2 of Chapter 31 is used in this step to estimate the average duration of an actuated phase. It distinguishes between actuated, noncoordinated, and coordinated phase types.

It is useful at this point to define the various terms that define phase duration. Some terms are specific to actuated operation; however, most constructs are equally applicable to pretimed operation.

The duration of an actuated phase is composed of five time periods. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the time required to clear the queue of vehicles. The third period represents the time the green indication is extended by randomly arriving vehicles. It ends when there is a gap in traffic (i.e., gap out) or the green extends to the maximum limit (i.e., max out). The fourth period represents the yellow change interval, and the fifth period represents the red clearance interval. The duration of an actuated phase is defined by Equation 18-13.

$$D_p = l_1 + g_s + g_e + Y + R_c$$

Equation 18-13

where

D_p = phase duration (s),

l_1 = start-up lost time = 2.0 (s),

g_s = queue service time (s),

g_e = green extension time (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

The relationship between the variables in Equation 18-13 is shown in Exhibit 18-14 by using a queue accumulation polygon.

Exhibit 18-14
Time Elements Influencing Actuated Phase Duration

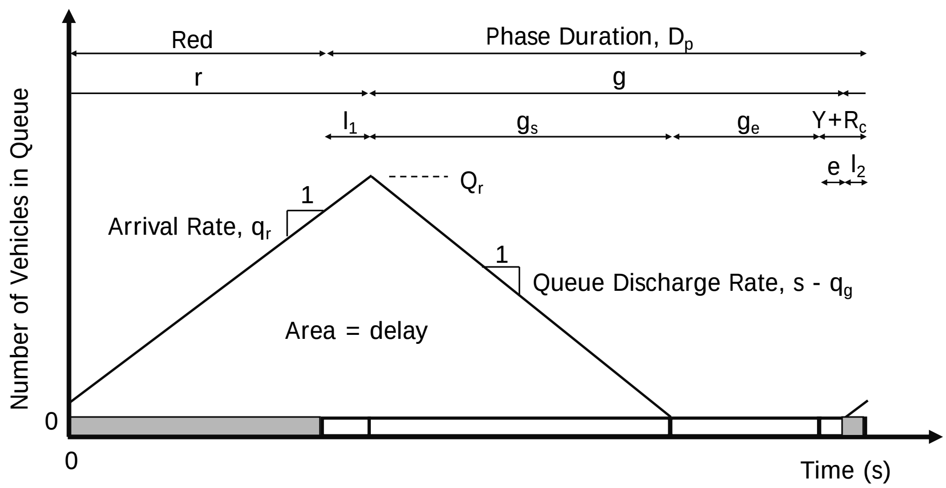


Exhibit 18-14 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the red interval ends. At this time, the queue begins to discharge at a rate equal to the saturation flow rate s less the arrival rate during green q_g. The queue clears g_s seconds after it first begins to discharge. Thereafter, random vehicle arrivals are detected and cause the green interval to be extended. Eventually, a gap occurs in traffic (or the maximum green limit is reached) and the green interval ends. The end of the green interval coincides with the end of the extension time g_e.

The effective green time for the phase is computed with the following equation:

Equation 18-14

$$g = D_p - l_1 - l_2 = g_s + g_e + e$$

where

l_2 = clearance lost time = $Y + R_c - e$ (s),

e = extension of effective green = 2.0 (s), and

all other variables are as previously defined.

Step 7: Determine Capacity and Volume-to-Capacity Ratio

Lane Group Volume-to-Capacity Ratio

The capacity of a given lane group serving one traffic movement, and for which there are no permitted left-turn movements, is defined by Equation 18-15.

$$c = N\,s\frac{g}{C}$$

Equation 18-15

where c is the capacity (veh/h) and other variables are as previously defined. This equation cannot be used to calculate the capacity of a shared-lane lane group or a lane group with permitted left-turn operation because these lane groups have other factors that affect their capacity. Chapter 31 provides a procedure for estimating the capacity of these types of lane groups.

The volume-to-capacity ratio for a lane group is defined as the ratio of the lane group volume and its capacity. It is computed using Equation 18-16.

$$X = \frac{v}{c}$$

Equation 18-16

where

X = volume-to-capacity ratio,

v = demand flow rate (veh/h), and

c = capacity (veh/h).

Critical Intersection Volume-to-Capacity Ratio

Another concept used for analyzing signalized intersections is the critical volume-to-capacity ratio X_c. This ratio is computed by using Equation 18-17 with Equation 18-18.

$$X_c = \left(\frac{C}{C-L}\right)\sum_{i \in ci} y_{c,i}$$

Equation 18-17

with

$$L = \sum_{i \in ci} l_{t,i}$$

Equation 18-18

where

X_c = critical intersection volume-to-capacity ratio,

C = cycle length (s),

$y_{c,i}$ = critical flow ratio for phase $i = v_i/(Ns_i)$,

$l_{t,i}$ = phase i lost time = $l_{1,i} + l_{2,i}$ (s),

ci = set of critical phases on the critical path, and

L = cycle lost time (s).

The summation term in each of these equations represents the sum of a specific variable for the set of critical phases. A critical phase is one phase of a set of phases that occur in sequence and whose combined flow ratio is the largest for

the signal cycle. The critical path and critical phases are identified by mapping traffic movements to a dual-ring phase diagram, as shown in Exhibit 18-3.

Equation 18-17 is based on the assumption that each critical phase has the same volume-to-capacity ratio and that this ratio is equal to the critical intersection volume-to-capacity ratio. This assumption is valid when the effective green duration for each critical phase i is proportional to $y_{c,i}/\Sigma(y_{c,i})$. When this assumption holds, the volume-to-capacity ratio for each noncritical phase is less than or equal to the critical intersection volume-to-capacity ratio.

Identifying Critical Lane Groups and Critical Flow Ratios

Calculation of the critical intersection volume-to-capacity ratio requires identification of the critical phases. This identification begins by mapping all traffic movements to a dual-ring diagram.

Next, the lane group flow ratio is computed for each lane group served by the phase. If a lane group is served only during one pretimed phase, then its flow ratio is computed as the lane group flow rate (per lane) divided by the lane group saturation flow rate [i.e., $v_i/(Ns_i)$]. If a lane group is served during multiple pretimed phases, then a flow ratio is computed for each phase. Specifically, the demand flow rate and saturation flow rate that occur during a given phase are used to compute the lane group flow ratio for that phase. For actuated phases, the flow ratio is computed only for those lane group-and-phase combinations in which the group's detectors actively extend the phase.

Next, the phase flow ratio is determined from the flow ratio of each lane group served during the phase. The phase flow ratio represents the largest flow ratio of all lane groups served.

Next, the diagram is evaluated to identify the critical phases. The phases that occur between one barrier pair are collectively evaluated to determine the critical phases. This evaluation begins with the pair in Ring 1 and proceeds to the pair in Ring 2. Each ring represents one possible critical path. The phase flow ratios are added for each phase pair in each ring. The larger of the two ring totals represents the critical path, and the corresponding phases represent the critical phases for the barrier pair.

Finally, the process is repeated for the phases between the other barrier pair. One critical flow rate is defined for each barrier pair by this process. These two values are then added to obtain the sum of the critical flow ratios used in Equation 18-17. The lost time associated with each of the critical phases is added to yield the cycle lost time L.

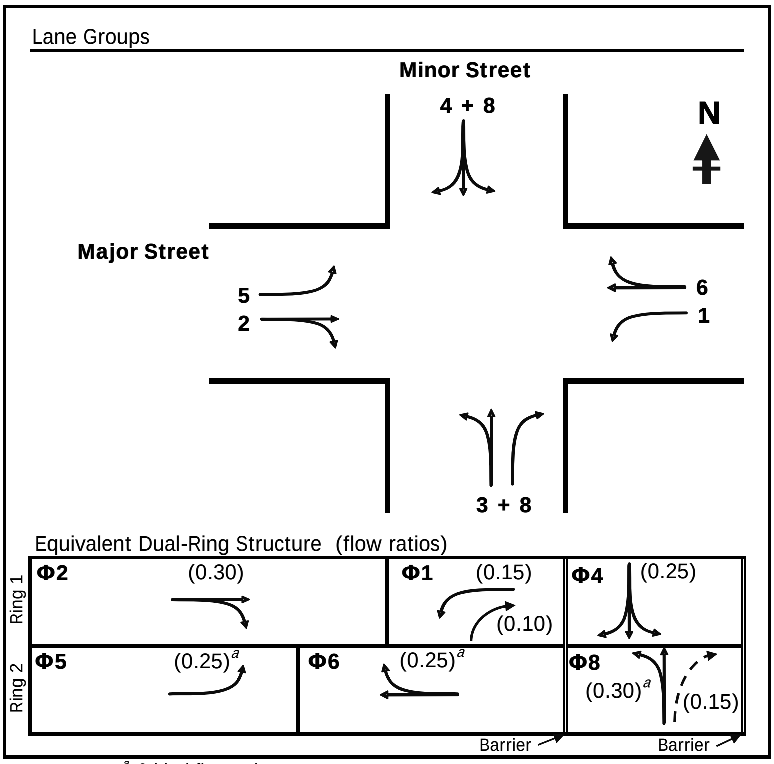
The procedure for the basic intersection case is explained in the next few paragraphs by using an example intersection. A variation of this procedure applies when protected-permitted left-turn operation is used with pretimed control. This variation is described after the basic case is described.

Basic Case

Consider a pretimed intersection with a lead–lag phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 18-15. The northbound right turn is provided an exclusive lane and a

green arrow indication that displays concurrently with the complementary left-turn phase on the major street. Each of the left-turn movements on the major street is served with a protected phase.

Exhibit 18-15
Critical Path Determination with Protected Left-Turn Phases



Note: [a] Critical flow ratio.

Phases 4 and 8 represent the only phases between the barrier pair serving the minor-street movements. Inspection of the flow ratios provided in the exhibit indicates that Phase 8 has two lane-group flow rates. The larger flow rate corresponds to the shared left-turn and through movement. Thus, the phase flow ratio for Phase 8 is 0.30. The phase flow ratio for Phase 4 is 0.25. Of the two phases, the largest phase flow ratio is that associated with Phase 8 (= 0.30), so it represents the critical phase for this barrier pair.

Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. A flow ratio is shown for the right-turn lane group in Phase 1 because the intersection has pretimed control. If the intersection was actuated, it is unlikely that the right-turn detection would be used to extend Phase 1, and the flow ratio for the right-turn lane group would not be considered in defining the phase flow ratio for Phase 1. Regardless, the phase flow ratio of Phase 1 is 0.15, on the basis of the left-turn lane group flow rate.

There are two possible critical paths through the major-street phase sequence—one path is associated with Phases 1 and 2 (i.e., Ring 1), and the other path is associated with Phases 5 and 6 (i.e., Ring 2). The total phase flow ratio for the Ring 1 path is 0.30 + 0.15, or 0.45. The total phase flow ratio for the Ring 2 path is 0.25 + 0.25 = 0.50. The latter total is larger and, hence, represents the

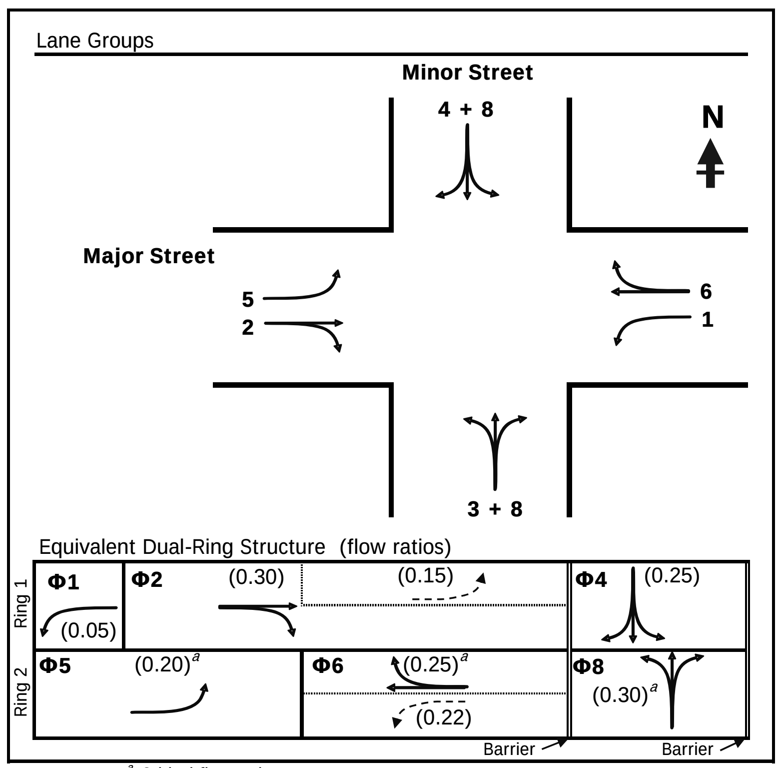
critical path. It identifies Phases 5 and 6 as the critical phases. Thus, the sum of critical flow ratios for the cycle is 0.80 (= 0.30 + 0.50).

One increment of phase lost time l_t is associated with each phase on the critical path. Thus, the cycle lost time L is computed as the sum of the lost time for each of Phases 5, 6, and 8.

Special Case: Pretimed Protected-Permitted Left-Turn Operation

Consider a pretimed intersection with a lead–lead phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 18-16. The left-turn movements on the major street operate in the protected-permitted mode. Phases 4 and 8 represent the only phases between one barrier pair. They serve the minor-street lane groups. By inspection of the flow ratios provided in the exhibit, Phase 8 has the highest flow ratio (= 0.30) of the two phases and represents the critical phase for this barrier pair.

Exhibit 18-16
Critical Path Determination with Protected-Permitted Left-Turn Operation



Note: [a] Critical flow ratio.

Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. Each left-turn lane group is shown to be served during two phases—once during the left-turn phase and once during the phase serving the adjacent through movement. The flow ratio for each of the four left-turn service periods is shown in Exhibit 18-16. The following rules define the possible critical paths through this phase sequence:

1. One path is associated with Phases 1 and 2 in Ring 1 (0.35 = 0.05 + 0.30).
2. One path is associated with Phases 5 and 6 in Ring 2 (0.45 = 0.20 + 0.25).

3. If a lead–lead or lag–lag phase sequence is used, then one path is associated with (*a*) the left-turn phase with the larger flow ratio and (*b*) the through phase that permissively serves the same left-turn lane group. Sum the protected and permitted left-turn flow ratios on this path (0.35 = 0.20 + 0.15).
4. If a lead–lag phase sequence is used, then one path is associated with (*a*) the leading left-turn phase, (*b*) the lagging left-turn phase, and (*c*) the *controlling* through phase (see discussion to follow). Sum the two protected left-turn flow ratios and the one controlling permitted left-turn flow ratio on this path.

If a lead–lag phase sequence is used, each of the through phases that permissively serve a left-turn lane group is considered in determining the *controlling* through phase. If both through phases have a permitted period, then there are two through phases to consider. The controlling through phase is that phase with the larger permitted left-turn flow ratio. For example, if Phase 1 were shown to lag Phase 2 in Exhibit 18-16, then Phase 6 would be the controlling through phase because the permitted left-turn flow ratio of 0.22 exceeds 0.15. The critical path for this phase sequence would be 0.47 (= 0.20 + 0.22 + 0.05).

The first three rules in the preceding list apply to the example intersection. The calculations are shown for each path in parentheses in the previous list of rules. The total flow ratio for the path in Ring 2 is largest (= 0.45) and, hence, represents the critical path. It identifies Phases 5 and 6 as the critical phases. Thus, the sum of critical flow ratios for the cycle is 0.75 (= 0.30 + 0.45).

If Rule 3 in the preceding list applies, then the only lost time incurred is the start-up lost time l_1 associated with the first critical phase and the clearance lost time l_2 associated with the second critical phase. If Rule 1, 2, or 4 applies, then one increment of phase lost time l_t is associated with each critical phase. Rule 2 applies for the example, so the cycle lost time L is computed as the sum of the lost time for each of Phases 5, 6, and 8.

Two flow ratios are associated with Phase 6 in this example. Both flow ratios are shown possibly to dictate the duration of Phase 6 (this condition does not hold for Phase 2 because of the timing of the left-turn phases). This condition is similar to that for the northbound right-turn movement in Phase 1 of Exhibit 18-15 and the treatment is the same. That is, both flow ratios are considered in defining the phase flow ratio for Phase 6.

This example is specific to pretimed control. If actuated control were used, then it is unlikely that the left-turn detection on the major street would be used to extend the through phases. In this situation, the flow ratio for the permitted left-turn lane group would not be considered in defining the phase flow ratio for the through phases (i.e., only the first two rules in the previous list would apply). In short, the analysis of protected-permitted left-turn operation with actuated control defaults to the basic case previously described.

Step 8: Determine Delay

The delay calculated in this step represents the average control delay experienced by all vehicles that arrive during the analysis period. It includes any delay incurred by these vehicles that are still in queue after the analysis period ends. The control delay for a given lane group is computed by using Equation 18-19.

Equation 18-19

$$d = d_1 + d_2 + d_3$$

where

d = control delay (s/veh),

d_1 = uniform delay (s/veh),

d_2 = incremental delay (s/veh), and

d_3 = initial queue delay (s/veh).

Concepts

Uniform Delay

Equation 18-20 represents one way to compute delay when arrivals are assumed to be random throughout the cycle. It also assumes one effective green period during the cycle and one saturation flow rate during this period. It is based on the first term of a delay equation presented elsewhere (*6*).

Equation 18-20

$$d_1 = \frac{0.5\,C\,(1 - g/C)^2}{1 - \left[\min(1, X) g/C\right]}$$

All variables are as previously defined. The delay calculation procedure used in this methodology is consistent with Equation 18-20. However, it removes the aforementioned assumptions to allow more accurate uniform delay estimates for progressed traffic movements, movements with multiple green periods, and movements with multiple saturation flow rates (e.g., protected-permitted turn movements). It is called the "incremental queue accumulation" procedure (*21, 22*).

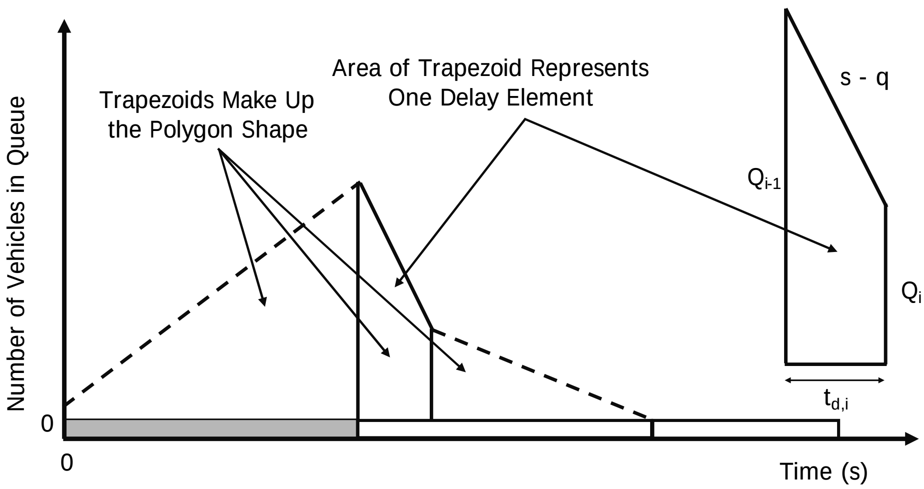
The incremental queue accumulation procedure models arrivals and departures as they occur during the average cycle. Specifically, it considers arrival rates and departure rates as they may occur during one or more effective green periods. The rates and resulting queue size can be shown in a queue accumulation polygon, such as that shown previously in Exhibit 18-14. The procedure decomposes the resulting polygon into an equivalent set of trapezoids or triangles for the purpose of delay estimation.

The key criterion for constructing a trapezoid or triangle is that the arrival and departure rates must be effectively constant during the associated time period. This process is illustrated in Exhibit 18-17 for a lane group having two different departure rates during the effective green period.

The delay associated with the cycle is determined by summing the area of the trapezoids or triangles that compose the polygon. The area of a given trapezoid or triangle is determined by first knowing the queue at the start of the

interval and then adding the number of arrivals and subtracting the number of departures during the specified time interval. The result of this calculation yields the number of vehicles in queue at the end of the interval. Equation 18-21 illustrates this calculation for interval *i*.

Exhibit 18-17
Decomposition of Queue Accumulation Polygon



Equation 18-21

$$Q_i = Q_{i-1} - (s/3{,}600 - q/N)\, t_{d,i} \quad \geq 0.0$$

where

Q_i = queue size at the end of interval *i* (veh),

q = arrival flow rate = v/3,600 (veh/s),

$t_{d,i}$ = duration of time interval *i* during which the arrival flow rate and saturation flow rate are constant (s), and

all other variables as previously defined.

Construction of the queue accumulation polygon requires converting all flow rate variables to common units of vehicles per second per lane. This conversion is implicit for all flow rate variables shown in exhibits here that depict a queue accumulation polygon.

Equation 18-22 is used to compute the *total* delay associated with a given trapezoid or triangle.

Equation 18-22

$$d_{T,i} = 0.5\,(Q_{i-1} + Q_i)\, t_{d,i}$$

where $d_{T,i}$ is the total delay associated with interval *i* (veh-s) and other variables are as previously defined. Total delay is computed for all intervals, added together, and the sum divided by the number of arrivals during the cycle (= qC) to estimate uniform delay in seconds per vehicle.

Construction of the queue accumulation polygon requires that the arrival flow rate not exceed the phase capacity. If the arrival flow rate exceeds capacity, then it is set to equal the capacity for the purpose of constructing the polygon. The queue can be assumed to equal zero at the end of the protected phase, and the polygon construction process begins at this point in the cycle. Once constructed, this assumption must be checked and, if the ending queue is not

zero, then a second polygon is constructed with this ending queue as the starting queue for the first interval.

Polygon construction requires identifying points in the cycle where one of the following two conditions applies:

- The departure rate changes (e.g., due to the start or end of effective green, a change in the saturation flow rate, depletion of the subject queue, depletion of the opposing queue, sneakers depart).
- The arrival rate changes (e.g., when a platoon arrival condition changes).

During the intervals of time between these points, the saturation flow rate and arrival flow rate are constant.

The determination of flow-rate-change points may require an iterative calculation process when the approach has shared lanes. For example, an analysis of the opposing through movement must be completed to determine the time this movement's queue clears and the subject left-turn lane group can begin its service period. This service period may, in turn, dictate when the permitted left-turn movements on the opposing approach may depart.

The procedure is based on defining arrival rate as having one of two flow states: an arrival rate during the green indication and an arrival rate during the red indication. Further information about when each of these rates applies is described in the discussion for platoon ratio in the required input data subsection. The proportion of vehicles arriving during the green indication P is used to compute the arrival flow rate during each flow state. The following equations can be used to compute these rates:

Equation 18-23

$$q_g = \frac{q\,P}{g/C}$$

and

Equation 18-24

$$q_r = \frac{q\,(1-P)}{1-g/C}$$

where

q_g = arrival flow rate during the effective green time (veh/s),

q_r = arrival flow rate during the effective red time (veh/s), and

all other variables as previously defined.

A more detailed description of the procedure for constructing a queue accumulation polygon for lane groups with various lane allocations and operating modes is provided in Chapter 31.

Incremental Delay

Incremental delay consists of two delay components. One component accounts for delay due to the effect of random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. This delay is evidenced by the occasional overflow queue at the end of the green interval (i.e., cycle failure). The second component accounts for delay due to a sustained oversaturation during

the analysis period. This delay occurs when aggregate demand during the analysis period exceeds aggregate capacity. It is sometimes referred to as the "deterministic" delay component and is shown as variable $d_{2,d}$ in Exhibit 18-18.

Exhibit 18-18
Cumulative Arrivals and Departures During an Oversaturated Analysis Period

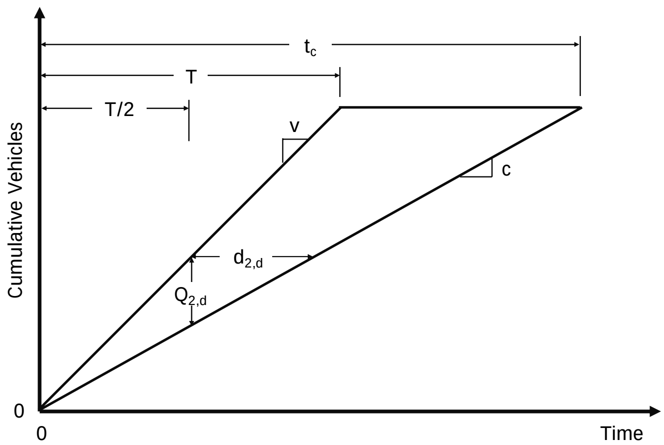


Exhibit 18-18 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate *v* during analysis period *T*, which has capacity *c*. The deterministic delay component is represented by the triangular area bounded by the thick line and is associated with an average delay per vehicle represented by the variable $d_{2,d}$. The last vehicle to arrive during the analysis period is shown to clear the queue t_c hours after the start of the analysis period. The average queue size associated with this delay is also shown in the exhibit as $Q_{2,d}$. The queue present at the end of the analysis period [= $T (v - c)$] is referred to as the *residual queue.*

Initial Queue Delay

The equation used to estimate incremental delay is based on the assumption that no initial queue is present at the start of the analysis period. The initial queue delay term accounts for the additional delay incurred due to an initial queue. This queue is a result of unmet demand in the previous time period. It does *not* include any vehicles that may be in queue due to random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. When a multiple-period analysis is undertaken, the initial queue for the second and subsequent analysis periods is equal to the residual queue from the previous analysis period.

Exhibit 18-19 illustrates the delay due to an initial queue as a trapezoid shape bounded by thick lines. The average delay per vehicle is represented by the variable d_3. The initial queue size is shown as Q_b vehicles. The duration of time during the analysis period for which the effect of the initial queue is still present is represented by the variable *t*. This duration is shown to equal the analysis period in Exhibit 18-19. However, it can be less than the analysis period duration for some lower-volume conditions.

Exhibit 18-19 illustrates the case in which the demand flow rate *v* exceeds the capacity *c* during the analysis period. In contrast, Exhibit 18-20 and Exhibit 18-21 illustrate alternative cases in which the demand flow rate is less than the capacity.

Exhibit 18-19
Initial Queue Delay with Increasing Queue Size

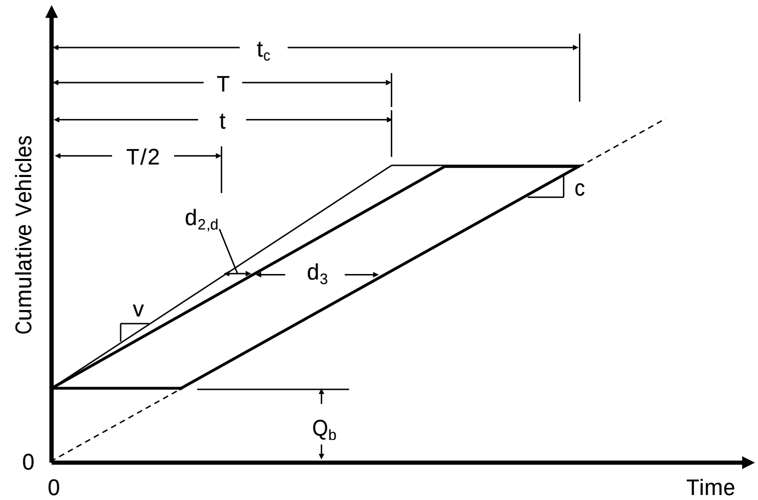


Exhibit 18-20
Initial Queue Delay with Decreasing Queue Size

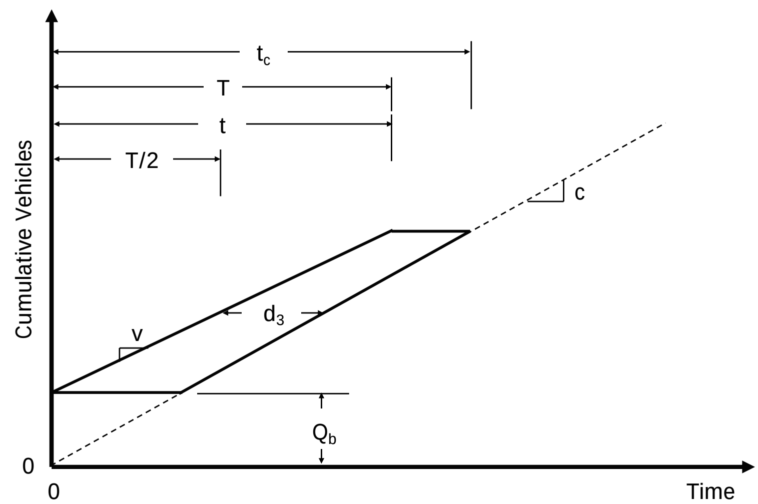
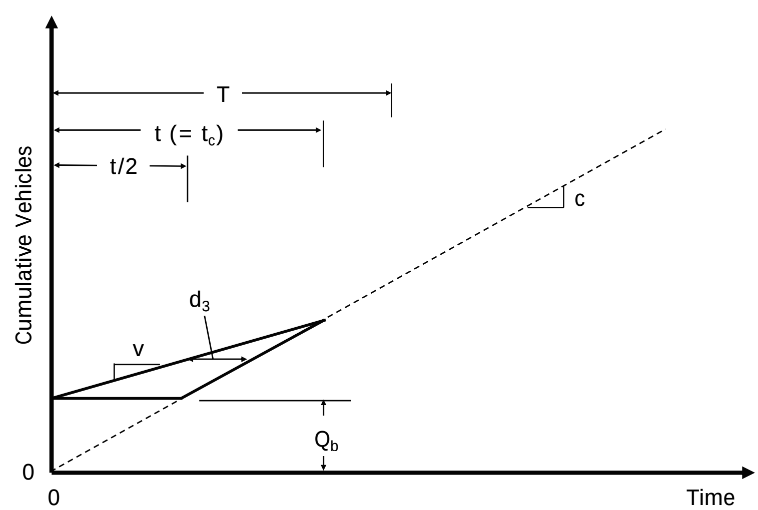


Exhibit 18-21
Initial Queue Delay with Queue Clearing



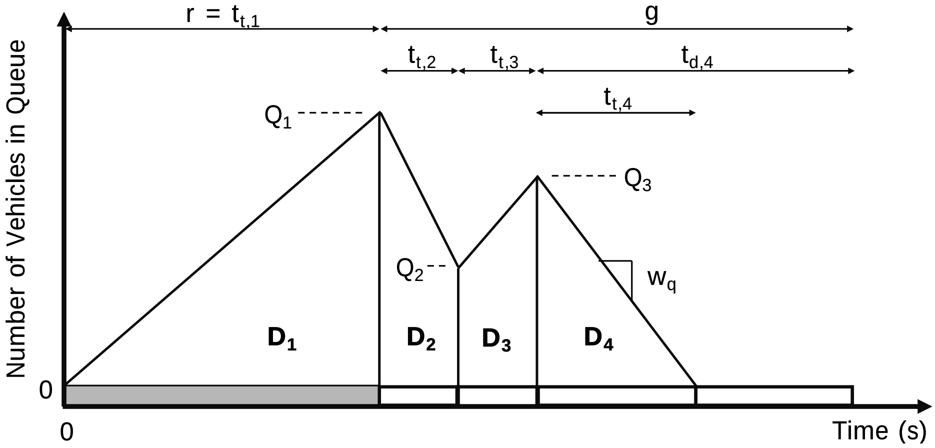
In this chapter, the phrase *initial queue* is always used in reference to the initial queue due to unmet demand in the previous time period. It *never* refers to vehicles in queue due to random, cycle-by-cycle fluctuations in demand.

The remainder of this step describes the procedure for computing the control delay for a lane group during a given analysis period. Chapter 31 describes a technique for measuring control delay in the field.

A. Compute Baseline Uniform Delay

Exhibit 18-14 was previously provided to illustrate a simple polygon for a lane group serving one traffic movement and for which there are no permitted or protected-permitted left-turn movements. Exhibit 18-22 is provided to illustrate delay calculation for a more complicated polygon shape. This particular polygon describes permitted left-turn operation from a shared lane for a specific combination of timing and volume conditions.

Exhibit 18-22
Polygon for Uniform Delay Calculation



The area bounded by the polygon represents the total delay incurred during the average cycle. The total delay is then divided by the number of arrivals per cycle to estimate the average uniform delay. These calculations are summarized in Equation 18-25, with Equation 18-26.

Equation 18-25

$$d_{1b} = \frac{0.5 \sum_{i=1} (Q_{i-1} + Q_i) t_{t,i}}{q\,C}$$

with

Equation 18-26

$$t_{t,i} = \min(t_{d,i}, Q_{i-1} / w_q)$$

where

d_{1b} = baseline uniform delay (s/veh),

$t_{t,i}$ = duration of trapezoid or triangle in interval *i* (s),

w_q = queue change rate (i.e., slope of the upper boundary of the trapezoid or triangle) (veh/s), and

all other variables as previously defined.

The summation term in Equation 18-25 includes all intervals for which there is a nonzero queue. In general, $t_{t,i}$ will equal the duration of the corresponding interval. However, during some intervals, the queue will dissipate and $t_{t,i}$ will only be as long as the time required for the queue to dissipate (= Q_{i-1}/w_q). This condition is shown to occur during Time Interval 4 in Exhibit 18-22.

The delay computed in this step is referred to as the *baseline uniform delay.* It may be adjusted in Step C if there is an initial queue that dissipates during the analysis period. The uniform delay to be used in Equation 18-19 is determined in this subsequent step.

B. Initial Queue Analysis

If an initial queue is present for any lane group at the intersection, then a second set of polygons needs to be constructed for each intersection lane group (in addition to those constructed for Step A). If no lane group has an initial queue, then this step is skipped.

At the start of this step, the initial queue that was input for each movement group needs to be converted to an initial queue for each lane group. When there is a one-to-one correlation between the movement group and the lane group, then the initial queue for the lane group equals the input initial queue for the movement group. When there is a shared lane on an approach that has another shared lane or additional through lanes, then the input initial queue needs to be distributed among the lane groups that serve the movements sharing the lane. Specifically, the initial queue for each lane group is estimated as being equal to the input initial queue multiplied by the number of lanes in the lane group and divided by the total number of shared and through lanes.

When the polygons are constructed in this step, lane groups with an initial queue will have their arrival flow rate set to equal the lane group capacity, regardless of their input arrival rate. The remaining lane groups will have their arrival flow rate set to equal the smaller of the input demand flow rate or the capacity. One polygon is constructed for each lane group, regardless of whether it has an initial queue.

The need for a second set of polygons stems from the influence one lane group often has on the operation of other lane groups. This influence is notably adverse when one or more lane groups are operating in a saturated state for a portion of the analysis period. If the saturated lane group represents a conflicting movement to a lane group that includes a permitted left-turn operation, then the left-turn lane group's operation will also be adversely affected for the same time period. Moreover, if the phase serving the lane group is actuated, then its capacity during the saturated state will be different from that of the subsequent unsaturated state. The following procedure is used to address this situation.

The duration of unmet demand is calculated in this step for each lane group with Equation 18-27 or Equation 18-28.

If $v \geq c_s$, then

Equation 18-27

$$t = T$$

If $v < c_s$, then

$$t = Q_b / (c_s - v) \quad \leq T$$

Equation 18-28

where

t = duration of unmet demand in the analysis period (h),

T = analysis period duration (h),

Q_b = initial queue at the start of the analysis period (veh),

v = demand flow rate (veh/h), and

c_s = saturated capacity (veh/h).

For this calculation, the saturated capacity c_s is equal to that obtained from the polygon constructed in this step and is reflective of the phase duration that is associated with saturated operation (due to the initial queue).

Next, the average duration of unmet demand is calculated with Equation 18-29.

$$t_a = \frac{1}{N_g} \sum_{i \in N_g} t_i$$

Equation 18-29

where

t_a = average duration of unmet demand in the analysis period (h), and

N_g = number of lane groups for which t exceeds 0.0 h.

The summation term in Equation 18-29 represents the sum of the t values for only those lane groups that have a value of t that exceeds 0.0 h. The average duration t_a is considered as a single representative value of t for all lane groups that do not have an initial queue.

The procedure described in Step A is repeated in this step to estimate the saturated uniform delay d_s for each lane group.

C. Compute Uniform Delay

If no lane group has an initial queue, then the uniform delay is equal to that computed in Step A (i.e., $d_1 = d_{1b}$). If an initial queue is present for any lane group at the intersection, then Equation 18-30 or Equation 18-31 is used to compute the uniform delay for each lane group.

If lane group i has an initial queue, then

$$d_{1,i} = d_{s,i} \frac{t_i}{T} + d_{1b,i} \frac{(T - t_i)}{T}$$

Equation 18-30

If lane group i does not have an initial queue, then

$$d_{1,i} = d_{s,i} \frac{t_a}{T} + d_{1b,i} \frac{(T - t_a)}{T}$$

Equation 18-31

where d_s is the saturated uniform delay (s/veh), t_i is the duration of unmet demand for lane group i in the analysis period (h), and other variables are as previously defined.

D. Compute Average Capacity

If no lane group has an initial queue, then the average lane group capacity c_A is equal to that computed in Step 7 (i.e., $c_A = c$). If an initial queue is present for any lane group at the intersection, then Equation 18-32 and Equation 18-33 are used to compute the average capacity for each lane group.

If lane group i has an initial queue, then

Equation 18-32
$$c_{A,i} = c_{s,i} \frac{t_i}{T} + c_i \frac{(T - t_i)}{T}$$

If lane group i does not have an initial queue, then

Equation 18-33
$$c_{A,i} = c_{s,i} \frac{t_a}{T} + c_i \frac{(T - t_a)}{T}$$

where c_A is the average capacity (veh/h) and other variables are as previously defined.

E. Compute Initial Queue Delay

If no lane group has an initial queue, then the initial queue delay d_3 is equal to 0.0 s/veh. If an initial queue is present for any lane group at the intersection, then Equation 18-34 through Equation 18-39 are used to compute the initial queue delay for each lane group.

Equation 18-34
$$d_3 = \frac{3{,}600}{v\,T}\left(t_A \frac{Q_b + Q_e - Q_{eo}}{2} + \frac{Q_e^2 - Q_{eo}^2}{2\,c_A} - \frac{Q_b^2}{2\,c_A} \right)$$

with

Equation 18-35
$$Q_e = Q_b + t_A (v - c_A)$$

If $v \geq c_A$, then

Equation 18-36
$$Q_{eo} = T(v - c_A)$$

Equation 18-37
$$t_A = T$$

If $v < c_A$, then

Equation 18-38
$$Q_{eo} = 0.0 \text{ veh}$$

Equation 18-39
$$t_A = Q_b / (c_A - v) \quad \leq T$$

where

t_A = adjusted duration of unmet demand in the analysis period (h),

Q_e = queue at the end of the analysis period (veh),

Q_{eo} = queue at the end of the analysis period when $v \geq c_A$ and $Q_b = 0.0$ (veh), and

other variables as previously defined.

The last vehicle that arrives to an overflow queue during the analysis period will clear the intersection at the time obtained with the following equation:

$$t_c = t_A + Q_e / c_A$$

Equation 18-40

where t_c is the queue clearing time (h) and other variables are as previously defined.

The queue clearing time is measured from the start of the analysis period to the time the last arriving vehicle clears the intersection.

F. Compute Incremental Delay Factor

The equation for computing incremental delay includes a variable that accounts for the effect of controller type on delay. This variable is referred to as the incremental delay factor *k*. It varies in value from 0.04 to 0.50. A factor value of 0.50 is recommended for pretimed phases, coordinated phases, and phases set to "recall-to-maximum."

An actuated phase has the ability to adapt its green interval duration to serve the demand on a cycle-by-cycle basis and, thereby, to minimize the frequency of cycle failure. Only when the green is extended to its maximum limit is this capability curtailed. This influence of actuated operation on delay is accounted for in Equation 18-41 through Equation 18-44.

$$k = (1 - 2k_{min})(v / c_a - 0.5) + k_{min} \quad \leq 0.50$$

Equation 18-41

with

$$k_{min} = -0.375 + 0.354\,PT - 0.0910\,PT^2 + 0.00889\,PT^3 \quad \geq 0.04$$

Equation 18-42

$$c_a = 3{,}600\,\frac{g_a\,s\,N}{C}$$

Equation 18-43

$$g_a = G_{max} + Y + R_c - l_1 - l_2$$

Equation 18-44

where

k = incremental delay factor,

c_a = available capacity for a lane group served by an actuated phase (veh/h),

k_{min} = minimum incremental delay factor, and

g_a = available effective green time (s).

All other variables are as previously defined. As indicated by this series of equations, the factor value depends on the maximum green setting and the passage time setting for the phase that controls the subject lane group. Research indicates that shorter passage times result in a lower value of *k* (and lower delay), provided that the passage time is not so short that the phase terminates before the queue is served (*11*).

G. Compute Incremental Delay

The incremental delay term accounts for delay due to random variation in the number of arrivals on a cycle-by-cycle basis. It also accounts for delay caused by demand exceeding capacity during the analysis period. The amount by which demand exceeds capacity during the analysis period is referred to here as unmet demand. The incremental delay equation was derived by using an assumption of

no initial queue due to unmet demand in the preceding analysis period. Equation 18-45, with Equation 18-46, is used to compute incremental delay.

Equation 18-45

$$d_2 = 900\, T \left[(X_A - 1) + \sqrt{(X_A - 1)^2 + \frac{8\, k\, I\, X_A}{c_A T}} \right]$$

with

Equation 18-46

$$X_A = v / c_A$$

where X_A is the average volume-to-capacity ratio and other variables are as previously defined. The incremental delay term is valid for all values of X_A, including highly oversaturated lane groups.

H. Compute Lane Group Control Delay

The uniform delay, incremental delay, and initial queue delay values computed in the previous steps are added (see Equation 18-19) to estimate the control delay for the subject lane group.

I. Compute Aggregated Delay Estimates

It is often desirable to compute the average control delay for the intersection approach. This aggregated delay represents a weighted average delay, where each lane group delay is weighted by the lane group demand flow rate. The approach control delay is computed with Equation 18-47.

Equation 18-47

$$d_{A,j} = \frac{\sum_{i=1}^{m_j} d_i v_i}{\sum_{i=1}^{m_j} v_i}$$

where

$d_{A,j}$ = approach control delay for approach j (s/veh),

d_i = control delay for lane group i (s/veh), and

m_j = number of lane groups on approach j.

All other variables are as previously defined. The summation terms in Equation 18-47 represent the sum over all lane groups on the subject approach.

Similarly, intersection control delay is computed with Equation 18-48.

Equation 18-48

$$d_I = \frac{\sum d_i v_i}{\sum v_i}$$

where d_I is the intersection control delay (s/veh) and all other variables are as previously defined. The summation terms in Equation 18-48 represent the sum over all lane groups at the subject intersection.

Step 9. Determine LOS

Exhibit 18-4 is used to determine the LOS for each lane group, each approach, and the intersection as a whole. LOS is an indication of the

acceptability of delay levels to motorists at the intersection. It can also indicate an unacceptable oversaturated operation for individual lane groups.

Step 10. Determine Queue Storage Ratio

A procedure is described in Chapter 31 for estimating the back-of-queue size and the queue storage ratio. The back of queue is the position of the vehicle stopped farthest from the stop line during the cycle as a consequence of the display of a red signal indication. The back-of-queue size depends on the arrival pattern of vehicles and on the number of vehicles that do not clear the intersection during the previous cycle.

The queue storage ratio represents the proportion of the available queue storage distance that is occupied at the point in the cycle when the back-of-queue position is reached. If this ratio exceeds 1.0, then the storage space will overflow and queued vehicles may block other vehicles from moving forward.

Extension to Multiple Time Periods

The 10-step sequence can be extended to analysis of consecutive time periods, each of duration T, and each having a fixed demand flow rate. The analysis is performed for each analysis period in sequence, as they occur in time. The initial queue Q_b for the second and subsequent periods is equal to the final queue Q_e from the previous period.

Typically, a multiple-time-period analysis would start with an undersaturated time period, desirably one when there is no initial queue for any intersection movement group. The demand flow rate for each period is a required input.

Interpretation of Results

The computations discussed in the previous steps result in the estimation of control delay and LOS for each lane group, for each approach, and for the intersection as a whole. They also produce a volume-to-capacity ratio for each lane group and a critical intersection volume-to-capacity ratio. This part provides some useful interpretations of these performance measures.

Level of Service

In general, LOS is an indication of the *general* acceptability of delay to drivers. In this regard, it should be remembered that what might be acceptable in a large city is not necessarily acceptable in a smaller city or rural area.

Intersection LOS must be interpreted with caution. It can suggest acceptable operation of the intersection when in reality certain lane groups (particularly those with lower volumes) are operating at an unacceptable LOS but are masked at the intersection level by the acceptable performance of higher-volume lane groups. The analyst should always verify that each lane group is providing acceptable operation and consider reporting the LOS for the poorest-performing lane group as a means of providing context to the interpretation of intersection LOS.

Volume-to-Capacity Ratio

In general, a volume-to-capacity ratio greater than 1.0 is an indication of actual or potential breakdown. In such cases, a multiple-period analysis is advised for this condition. This analysis would encompass all consecutive periods in which a residual queue is present.

The critical intersection volume-to-capacity ratio from Equation 18-17 is useful in evaluating the intersection from a capacity-only perspective. It is possible to have a critical intersection volume-to-capacity ratio of less than 1.0 and still have individual movements oversaturated within the signal cycle. If this situation occurs, then the cycle time is generally not appropriately allocated among the phases. Reallocation of the cycle time should be considered, where additional time is given to the phases serving those lane groups with a volume-to-capacity ratio greater than 1.0.

A critical intersection volume-to-capacity ratio greater than 1.0 indicates that the overall signal timing and geometric design provide inadequate capacity for the given demand flows. Improvements that might be considered include the following:

- Basic changes in intersection geometry (i.e., change in the number or use of lanes),
- Increase in signal cycle length if it is determined to be too short, and
- Changes in signal phase sequence or timing.

Local guidelines should always be consulted before potential improvements are developed.

Fully actuated control is intended to allocate cycle time dynamically to movements on the basis of demand and, thereby, maintain efficient operation on a cycle-by-cycle basis. The critical intersection volume-to-capacity ratio can provide an indication of this efficiency. In general, this ratio will vary between 0.85 and 0.95 for most actuated intersections, with lower values in this range more common for intersections having multiple detectors in the through traffic lanes. A ratio less than 0.85 may be an indication of excessive green extension by random arrivals, and the analyst may consider reducing passage time, minimum green, or both. A ratio that is more than 0.95 may be an indication of frequent phase termination by max out and limited ability of the controller to reallocate cycle time dynamically on the basis of detected demand. Increasing the maximum green may improve operation in some instances; however, it may also degrade operation when phase flow rates vary widely (because green extension is based on total flow rate served by the phase, not flow rate per lane).

For semiactuated and coordinated-actuated control, the critical intersection volume-to-capacity ratio can vary widely because of the nonactuated nature of some phases. The duration of these phases may not be directly related to their associated demand; instead, it may be dictated by coordination timing or the demand for the other phases. A critical intersection volume-to-capacity ratio that exceeds 0.95 has the same interpretation as offered previously for fully actuated control.

The critical intersection volume-to-capacity ratio can be misleading when it is used to evaluate the overall sufficiency of the intersection geometry, as is often required in planning applications. The problem is that low flow rates dictate the need for short cycle lengths to minimize delay. Yet, Equation 18-17 indicates that the desired shorter cycle length produces a higher volume-to-capacity ratio. Therefore, a relatively large value of X_c (provided that it is less than 1.0) is not a certain indication of poor operation. Rather, it means that closer attention must be paid to the adequacy of phase duration and queue size, especially for the critical phases.

Volume-to-Capacity Ratio and Delay Combinations

In some cases, delay is high even when the volume-to-capacity ratio is low. In these situations, poor progression, a notably long cycle length, or an inefficient phase plan is generally the cause. When the intersection is part of a coordinated system, the cycle length is determined by system considerations, and alterations at individual intersections may not be practical.

It is possible that delay is at acceptable levels even when the volume-to-capacity ratio is high. This situation can occur when some combination of the following conditions exists: the cycle length is relatively short, the analysis period is short, the lane group capacity is high, and there is no initial queue. If a residual queue is created in this scenario, then the conduct of a multiple-period analysis is necessary to gain a true picture of the delay.

When both delay levels and volume-to-capacity ratios are unacceptably high, the situation is critical. In such situations, the delay may increase rapidly with small changes in demand. The full range of potential geometric and signal design changes should be considered in the search for improvements.

In summary, unacceptable delay can exist when capacity is a problem as well as when capacity is adequate. Further, acceptable delay levels do not automatically ensure that capacity is sufficient. Delay and capacity are complex variables that are influenced by a wide range of traffic, roadway, and signalization conditions. The methodology presented here can be used to estimate these performance measures, identify possible problems, and assist in developing alternative improvements.

PEDESTRIAN MODE

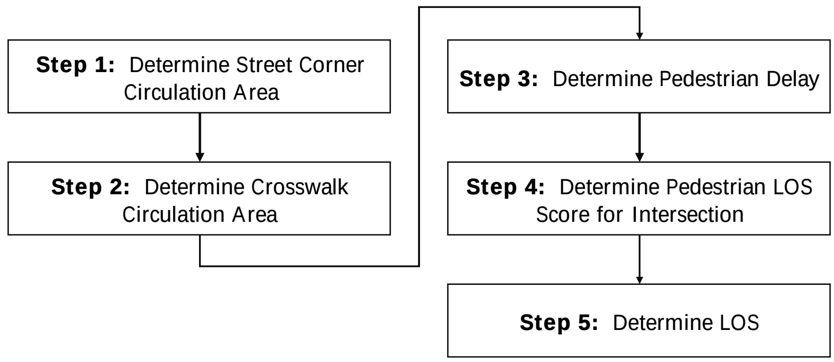
This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to pedestrians.

Intersection performance is separately evaluated for each crosswalk and intersection corner with this methodology. *Unless otherwise stated, all variables identified in this subsection are specific to one crosswalk and one corner.* A crosswalk is assumed to exist across each intersection leg unless crossing is specifically prohibited by local ordinance (and signed to this effect).

The methodology is focused on the analysis of signalized intersection performance. Chapter 17, Urban Street Segments, and Chapter 19, Two-Way STOP-Controlled Intersections, describe methodologies for evaluating the performance of these system elements with respect to the pedestrian mode.

The pedestrian methodology is applied through a series of five steps that determine the pedestrian LOS for a crosswalk and associated corners. These steps are illustrated in Exhibit 18-23.

Exhibit 18-23
Pedestrian Methodology for Signalized Intersections



Concepts

Performance Measures

The methodology provides a variety of measures for evaluating intersection performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip through the intersection. Performance measures that are estimated include the following:

- Corner circulation area,
- Crosswalk circulation area,
- Pedestrian delay, and
- Pedestrian LOS score.

The first two performance measures listed are based on the concept of "circulation area." One measure is used to evaluate the circulation area provided to pedestrians while they wait at the corner. Another measure is used to evaluate the area provided while the pedestrian is crossing in the crosswalk. Circulation area describes the space available to the average pedestrian. A larger area is more desirable from the pedestrian perspective. Exhibit 18-24 can be used to evaluate intersection performance from a circulation-area perspective.

Exhibit 18-24
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p)	Description
>60	Ability to move in desired path, no need to alter movements
>40–60	Occasional need to adjust path to avoid conflicts
>24–40	Frequent need to adjust path to avoid conflicts
>15–24	Speed and ability to pass slower pedestrians restricted
>8–15	Speed restricted, very limited ability to pass slower pedestrians
≤8	Speed severely restricted, frequent contact with other users

Pedestrian delay represents the average time a pedestrian waits for a legal opportunity to cross an intersection leg. The LOS score is an indication of the typical pedestrian's perception of the overall crossing experience.

Flow Conditions

Exhibit 18-25 and Exhibit 18-26 show the variables considered when one corner and its two crosswalks are evaluated. Two flow conditions are illustrated.

Condition 1 corresponds to the minor-street crossing that occurs during the major-street through phase. The pedestrians who desire to cross the major street must wait at the corner. Condition 2 corresponds to the major-street crossing that occurs during the minor-street through phase. For this condition, the pedestrians who desire to cross the minor street wait at the corner.

Exhibit 18-25
Condition 1: Minor-Street Crossing

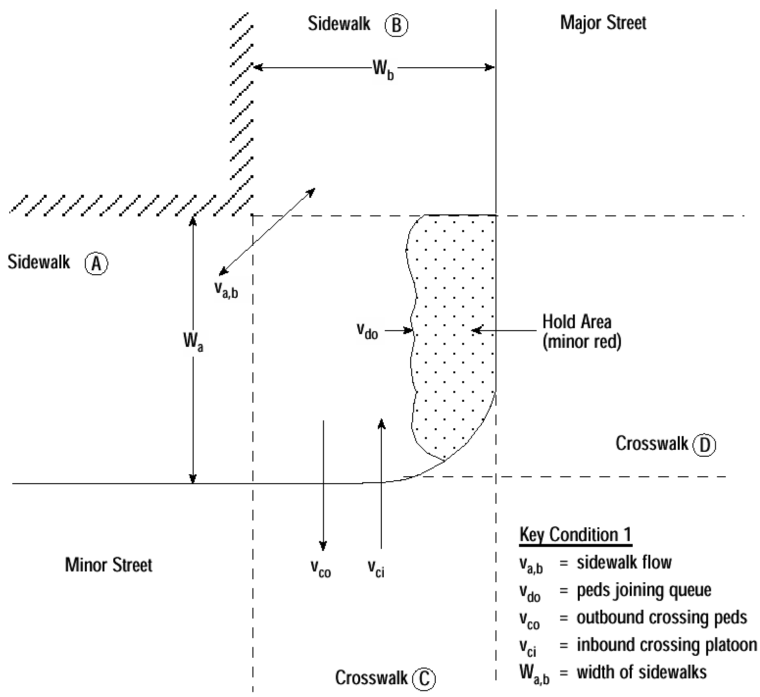
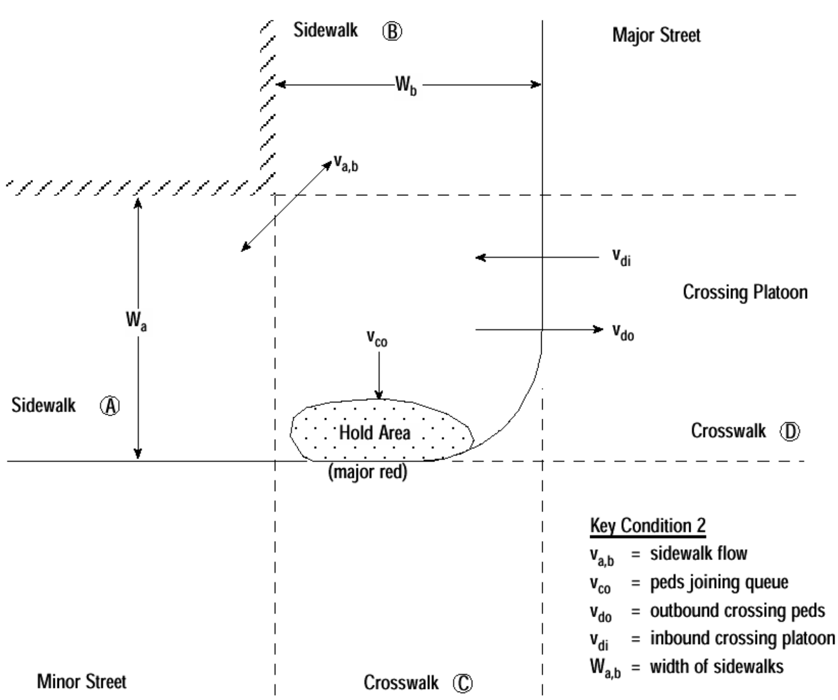


Exhibit 18-26
Condition 2: Major-Street Crossing



Effective Walk Time

Research indicates that, at intersections with pedestrian signal heads, pedestrians typically continue to enter the intersection during the first few seconds of the pedestrian clear interval (*26*, *28*). This behavior effectively increases the effective walk time available to pedestrians. A conservative estimate of this additional walk time is 4.0 s (*26*). A nonzero value for this additional time implies that some pedestrians are initiating their crossing during the flashing DON'T WALK indication.

The following guidance is provided to estimate the effective walk time on the basis of the aforementioned research findings. If the phase providing service to the pedestrians is either (*a*) actuated with a pedestrian signal head and rest-in-walk *not* enabled or (*b*) pretimed with a pedestrian signal head, then

Equation 18-49

$$g_{\text{Walk}} = Walk + 4.0$$

If the phase providing service to the pedestrians is actuated with a pedestrian signal head and rest-in-walk enabled, then

Equation 18-50

$$g_{\text{Walk}} = D_p - Y - R_c - PC + 4.0$$

Otherwise (i.e., no pedestrian signal head)

Equation 18-51

$$g_{\text{Walk}} = D_p - Y - R_c$$

where

g_{Walk} = effective walk time (s),

$Walk$ = pedestrian walk setting (s),

PC = pedestrian clear setting (s),

D_p = phase duration (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

The aforementioned research indicates that the effective walk time estimated with Equation 18-49 or Equation 18-50 can vary widely among intersections. At a given intersection, the additional walk time can vary from 0.0 s to an amount equal to the pedestrian clear interval. The amount of additional walk time used by pedestrians depends on many factors, including the extent of pedestrian delay, vehicular volume, level of enforcement, and presence of countdown pedestrian signal heads.

The effective walk time estimated with Equation 18-49 or Equation 18-50 is considered to be directly applicable to design or planning analyses because it is conservative in the amount of additional walk time that it includes. A larger value of effective walk time may be applicable to an operational analysis if (*a*) field observation or experience indicates such a value would be consistent with actual pedestrian use of the flashing DON'T WALK indication; (*b*) an accurate estimate of pedestrian delay or queue size is desired; and (*c*) the predicted performance estimates are understood to reflect some illegal pedestrian behavior, possibly in response to constrained spaces or inadequate signal timing.

Step 1: Determine Street Corner Circulation Area

This step describes a procedure for evaluating the performance of one intersection corner. It is repeated for each intersection corner of interest.

The analysis of circulation area at the street corners and in the crosswalks compares available time and space with pedestrian demand. The product of time and space is the critical parameter. It combines the constraints of physical design (which limits available space) and signal operation (which limits available time). This parameter is hereafter referred to as "time–space."

A. Compute Available Time–Space

The total time–space available for circulation and queuing in the intersection corner equals the product of the net corner area and the cycle length C. Equation 18-52 is used to compute time–space available at an intersection corner. Exhibit 18-10 identifies the variables used in the equation.

Equation 18-52

$$TS_{corner} = C\,(W_a W_b - 0.215\,R^2)$$

where

TS_{corner} = available corner time–space (ft²-s),

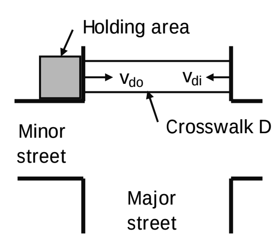
C = cycle length (s),

W_a = total walkway width of Sidewalk A (ft),

W_b = total walkway width of Sidewalk B (ft), and

R = radius of corner curb (ft).

If the corner curb radius is larger than either W_a or W_b, then the variable R in Equation 18-52 should equal the smaller of W_a or W_b.



B. Compute Holding-Area Waiting Time

The average pedestrian holding time represents the average time that pedestrians wait to cross the street when departing from the subject corner. The equation for computing this time is based on the assumption that pedestrian arrivals are uniformly distributed during the cycle. For Condition 1, as shown in Exhibit 18-25, Equation 18-53 and Equation 18-54 are used to compute holding-area time for pedestrians waiting to cross the major street.

Equation 18-53

$$Q_{tdo} = \frac{N_{do}(C - g_{\text{Walk},mi})^2}{2C}$$

with

Equation 18-54

$$N_{do} = \frac{v_{do}}{3{,}600}\,C$$

where

Q_{tdo} = total time spent by pedestrians waiting to cross the major street during one cycle (p-s),

N_{do} = number of pedestrians arriving at the corner each cycle to cross the major street (p),

$g_{\text{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s),

C = cycle length (s), and

v_{do} = flow rate of pedestrians arriving at the corner to cross the major street (p/h).

If the phase providing service to the pedestrians is either (*a*) actuated with a pedestrian signal head and rest-in-walk *not* enabled or (*b*) pretimed with a pedestrian signal head, then

Equation 18-55

$$g_{\text{Walk},mi} = Walk_{mi} + 4.0$$

If the phase providing service to the pedestrians is actuated with a pedestrian signal head and rest-in-walk enabled, then

Equation 18-56

$$g_{\text{Walk},mi} = D_{p,mi} - Y_{mi} - R_{c,mi} - PC_{mi} + 4.0$$

Otherwise (i.e., no pedestrian signal head)

Equation 18-57

$$g_{\text{Walk},mi} = D_{p,mi} - Y_{mi} - R_{c,mi}$$

where

$g_{\text{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s),

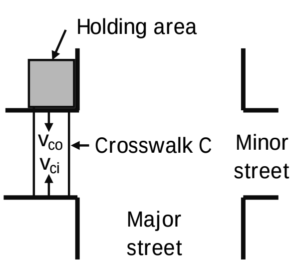
$Walk_{mi}$ = pedestrian walk setting for the phase serving the minor-street through movement (s),

PC_{mi} = pedestrian clear setting for the phase serving the minor-street through movement (s),

$D_{p,mi}$ = duration of the phase serving the minor-street through movement (s),

Y_{mi} = yellow change interval of the phase serving the minor-street through movement (s), and

$R_{c,mi}$ = red clearance interval of the phase serving the minor-street through movement (s).



For Condition 2, the previous three equations are repeated to compute the holding-area time for pedestrians waiting to cross the minor street Q_{tco}. For this application, the subscript letters "*do*" are replaced with the letters "*co*" to denote the pedestrians arriving at the corner to cross in Crosswalk C. Similarly, the subscript letters "*mi*" are replaced with "*mj*" to denote signal timing variables associated with the phase serving the major-street through movement.

C. Compute Circulation Time–Space

The time–space available for circulating pedestrians equals the total available time–space minus the time–space occupied by the pedestrians waiting to cross. The latter value equals the product of the total waiting time and the area used by waiting pedestrians (= 5.0 ft²/p). Equation 18-58 is used to compute the time–space available for circulating pedestrians.

Equation 18-58

$$TS_c = TS_{corner} - \left[5.0\,(Q_{tdo} + Q_{tco})\right]$$

where TS_c is the time–space available for circulating pedestrians (ft^2-s) and other variables are as previously defined.

D. Compute Pedestrian Corner Circulation Area

The space required for circulating pedestrians is computed by dividing the time–space available for circulating pedestrians by the time that pedestrians consume walking through the corner area. The latter quantity equals the total circulation volume multiplied by the assumed average circulation time (= 4.0 s). Equation 18-59, with Equation 18-60, is used to compute corner circulation area.

Equation 18-59

$$M_{\text{corner}} = \frac{TS_c}{4.0\, N_{tot}}$$

with

Equation 18-60

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600} C$$

where

M_{corner} = corner circulation area per pedestrian (ft^2/p),

N_{tot} = total number of circulating pedestrians that arrive each cycle (p),

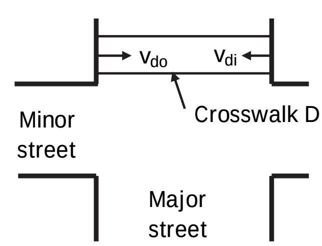
v_{ci} = flow rate of pedestrians arriving at the corner after crossing the minor street (p/h),

v_{co} = flow rate of pedestrians arriving at the corner to cross the minor street (p/h),

v_{di} = flow rate of pedestrians arriving at the corner after crossing the major street (p/h), and

$v_{a,b}$ = flow rate of pedestrians traveling through the corner from Sidewalk A to Sidewalk B, or vice versa (p/h).

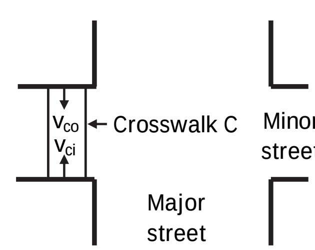
Other variables are as previously defined. The circulation area obtained from Equation 18-59 can be compared with the ranges provided in Exhibit 18-24 to make some judgments about the performance of the subject intersection corner.



Step 2: Determine Crosswalk Circulation Area

This step describes a procedure for evaluating the performance of one crosswalk. It is repeated for each crosswalk of interest.

The procedure to follow describes the evaluation of Crosswalk D in Exhibit 18-26 (i.e., a crosswalk across the major street). The procedure is repeated to evaluate Crosswalk C in Exhibit 18-25. For the second application, the subscript letters "*do*" and "*di*" are replaced with the letters "*co*" and "*ci*," respectively, to denote the pedestrians associated with Crosswalk C. Similarly, the subscript letter "*d*" is replaced with the letter "*c*" to denote the length and width of Crosswalk C. Also, the subscript letters "*mi*" are replaced with "*mj*" to denote signal timing variables associated with the phase serving the major-street through movement.



The recommended walking speeds reflect average (50th percentile) walking speeds for the purposes of calculating LOS. Traffic signal timing for pedestrians is typically based on a 15th percentile walking speed.

A. Establish Walking Speed

The average pedestrian walking speed S_p is needed to evaluate corner and crosswalk performance. Research indicates that the walking speed is influenced by pedestrian age and sidewalk grade (*26*). If 0% to 20% of pedestrians traveling along the subject segment are elderly (i.e., 65 years of age or older), an average walking speed of 4.0 ft/s is recommended for intersection evaluation. If more than 20% of all pedestrians are elderly, an average walking speed of 3.3 ft/s is recommended. In addition, an upgrade of 10% or greater reduces walking speed by 0.3 ft/s.

B. Compute Available Time–Space

Equation 18-61 is used to compute the time–space available in the crosswalk.

Equation 18-61

$$TS_{cw} = L_d W_d \, g_{\mathrm{Walk},mi}$$

where

TS_{cw} = available crosswalk time–space (ft^2-s),

L_d = length of Crosswalk D (ft),

W_d = effective width of Crosswalk D (ft), and

$g_{\mathrm{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s).

C. Compute Effective Available Time–Space

The available crosswalk time–space is adjusted in this step to account for the effect turning vehicles have on pedestrians. This adjustment is based on the assumed occupancy of a vehicle in the crosswalk. The vehicle occupancy is computed as the product of vehicle swept-path, crosswalk width, and the time the vehicle preempts this space. Equation 18-62 through Equation 18-64 are used for this purpose.

Equation 18-62

$$TS_{cw}^{*} = TS_{cw} - TS_{tv}$$

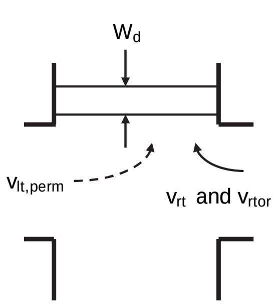
with

Equation 18-63

$$TS_{tv} = 40\, N_{tv} W_d$$

Equation 18-64

$$N_{tv} = \frac{v_{lt,perm} + v_{rt} - v_{rtor}}{3{,}600} C$$



where

TS_{cw}^{*} = effective available crosswalk time–space (ft^2-s),

TS_{tv} = time–space occupied by turning vehicles (ft^2-s),

N_{tv} = number of turning vehicles during the walk and pedestrian clear intervals (veh),

$v_{lt,perm}$ = permitted left-turn demand flow rate (veh/h),

v_{rt} = right-turn demand flow rate (veh/h), and

v_{rtor} = right-turn-on-red flow rate (veh/h).

Other variables are as previously defined. The constant 40 in Equation 18-63 represents the product of the swept-path for most vehicles (= 8 ft) and the time that a turning vehicle occupies the crosswalk (= 5 s). The left-turn and right-turn flow rates used in Equation 18-64 are those associated with movements that receive a green indication concurrently with the subject pedestrian crossing *and* turn across the subject crosswalk.

D. Compute Pedestrian Service Time

Total service time is computed with either Equation 18-65 or Equation 18-66, depending on the crosswalk width, along with Equation 18-67. This time represents the elapsed time starting with the first pedestrian's departure from the corner to the last pedestrian's arrival at the far side of the crosswalk. In this manner, it accounts for platoon size in the service time (*29*).

If crosswalk width W_d is greater than 10 ft, then

$$t_{ps,do} = 3.2 + \frac{L_d}{S_p} + 2.7 \frac{N_{ped,do}}{W_d}$$

Equation 18-65

If crosswalk width W_d is less than or equal to 10 ft, then

$$t_{ps,do} = 3.2 + \frac{L_d}{S_p} + 0.27\ N_{ped,do}$$

Equation 18-66

with

$$N_{ped,do} = N_{do} \frac{C - g_{\text{Walk},mi}}{C}$$

Equation 18-67

where

$t_{ps,do}$ = service time for pedestrians that arrive at the corner to cross the major street (s),

$N_{ped,do}$ = number of pedestrians waiting at the corner to cross the major street (p), and

other variables are as previously defined.

Equation 18-67 provides an estimate of the number of pedestrians who cross as a group following the presentation of the WALK indication (or green indication, if pedestrian signal heads are not provided). It is also used to compute $N_{ped,di}$ for the other travel direction in the same crosswalk (using N_{di}, as defined below). Finally, Equation 18-65 or Equation 18-66 is used to compute the service time for pedestrians who arrive at the subject corner having waited on the other corner before crossing the major street $t_{ps,di}$ (using $N_{ped,di}$).

E. Compute Crosswalk Occupancy Time

The total crosswalk occupancy time is computed as a product of the pedestrian service time and the number of pedestrians using the crosswalk during one signal cycle. Equation 18-68 is used, with Equation 18-69 and results from previous steps, for the computation.

Equation 18-68

$$T_{occ} = t_{ps,do} N_{do} + t_{ps,di} N_{di}$$

with

Equation 18-69

$$N_{di} = \frac{v_{di}}{3{,}600} C$$

where

T_{occ} = crosswalk occupancy time (p-s), and

N_{di} = number of pedestrians arriving at the corner each cycle having crossed the major street (p).

Other variables are as previously defined.

F. Compute Pedestrian Crosswalk Circulation Area

The circulation space provided for each pedestrian is determined by dividing the time–space available for crossing by the total occupancy time, as shown in Equation 18-70.

Equation 18-70

$$M_{cw} = \frac{TS^{*}_{cw}}{T_{occ}}$$

where M_{cw} is the crosswalk circulation area per pedestrian (ft^2/p) and other variables are as previously defined.

The circulation area obtained from Equation 18-70 can be compared with the ranges provided in Exhibit 18-24 to make some judgments about the performance of the subject-intersection crosswalk (for the specified direction of travel). For a complete picture of the subject crosswalk's performance, the procedure described in this step should be repeated for the other direction of travel along the crosswalk (i.e., by using the other corner associated with the crosswalk as the point of reference).

Step 3: Determine Pedestrian Delay

This step describes a procedure for evaluating the performance of a crosswalk at the intersection. It is repeated for each crosswalk of interest.

The discussion that follows describes the evaluation of Crosswalk D shown in Exhibit 18-26. The procedure is applied again to evaluate Crosswalk C shown in Exhibit 18-25. For the second application, the subscript letters "*mi*" are replaced with "*mj*" to denote signal timing variables associated with the phase serving the major-street through movement.

The pedestrian delay while waiting to cross the major street is computed with Equation 18-71.

Equation 18-71

$$d_p = \frac{(C - g_{\text{Walk},mi})^2}{2C}$$

where d_p is pedestrian delay (s/p) and other variables are as previously defined.

The delay obtained from Equation 18-71 applies equally to both directions of travel along the crosswalk.

Research indicates that average pedestrian delay at signalized intersection crossings is not constrained by capacity, even when pedestrian flow rates reach 5,000 p/h (*26*). For this reason, delay due to oversaturated conditions is not included in the value obtained from Equation 18-71.

If the subject crosswalk is closed, then the pedestrian delay d_p is estimated as the value obtained from Equation 18-71 for the subject crosswalk, plus two increments of the delay from this equation when applied to the perpendicular crosswalk. This adjustment reflects the additional delay pedestrians incur when crossing the other three legs of the intersection so that they can continue walking in the desired direction.

The pedestrian delay computed in this step can be used to make some judgment about pedestrian compliance. In general, pedestrians become impatient when they experience delays in excess of 30 s/p, and there is a high likelihood of their not complying with the signal indication (*30*). In contrast, pedestrians are very likely to comply with the signal indication if their expected delay is less than 10 s/p.

Step 4: Determine Pedestrian LOS Score for Intersection

This step describes a procedure for evaluating the performance of one crosswalk. It is repeated for each crosswalk of interest.

The procedure to follow describes the evaluation of Crosswalk D in Exhibit 18-26. The procedure is repeated to evaluate Crosswalk C in Exhibit 18-25. For the second application, the subscript letter "*d*" is replaced with the letter "*c*" to denote the length and width of Crosswalk C. Also, the subscript letters "*mj*" are replaced with "*mi*" to denote variables associated with the minor street.

The pedestrian LOS score for the intersection $I_{p,int}$ is calculated by using Equation 18-72 through Equation 18-77.

$$I_{p,int} = 0.5997 + F_w + F_v + F_S + F_{\text{delay}}$$

Equation 18-72

with

$$F_w = 0.681\left(N_d\right)^{0.514}$$

Equation 18-73

$$F_v = 0.00569\left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,d}\left(0.0027\ n_{15,mj} - 0.1946\right)$$

Equation 18-74

$$F_S = 0.00013\ n_{15,mj}\ S_{85,mj}$$

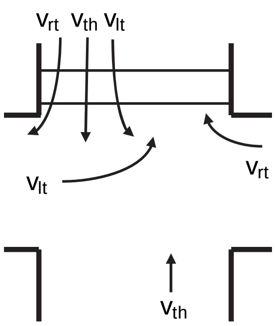
Equation 18-75

$$F_{\text{delay}} = 0.0401\ \ln(d_{p,d})$$

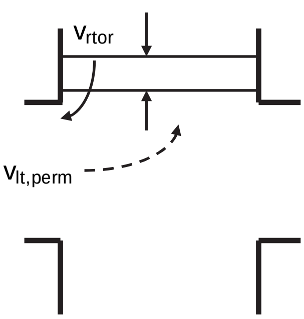
Equation 18-76

$$n_{15,mj} = \frac{0.25}{N_d}\sum_{i \in m_d} v_i$$

Equation 18-77



Movements crossing Crosswalk D



where

$I_{p,int}$ = pedestrian LOS score for intersection,

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_{delay} = pedestrian delay adjustment factor,

$\ln(x)$ = natural logarithm of x,

N_d = number of traffic lanes crossed when traversing Crosswalk D (ln),

$N_{rtci,d}$ = number of right-turn channelizing islands along Crosswalk D,

$n_{15,mj}$ = count of vehicles traveling on the major street during a 15-min period (veh/ln),

$S_{85,mj}$ = 85th percentile speed at a midsegment location on the major street (mi/h),

$d_{p,d}$ = pedestrian delay when traversing Crosswalk D (s/p),

v_i = demand flow rate for movement i (veh/h), and

m_d = set of all automobile movements that cross Crosswalk D (see figure in margin).

The left-turn flow rate $v_{lt,perm}$ used in Equation 18-74 is that associated with the left-turn movement that receives a green indication concurrently with the subject pedestrian crossing *and* turns across the subject crosswalk. The RTOR flow rate v_{rtor} is that associated with the approach being crossed and that turns across the subject crosswalk. It is not the same v_{rtor} used in Equation 18-64.

The pedestrian LOS score obtained from this equation applies equally to both directions of travel along the crosswalk.

The variable for "number of right-turn channelizing islands" N_{rtci} is an integer with a value of 0, 1, or 2.

Step 5: Determine LOS

This step describes a process for determining the LOS of one crosswalk. It is repeated for each crosswalk of interest.

The pedestrian LOS is determined by using the pedestrian LOS score from Step 4. This performance measure is compared with the thresholds in Exhibit 18-5 to determine the LOS for the subject crosswalk.

BICYCLE MODE

This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to bicyclists.

Intersection performance is evaluated separately for each intersection approach. *Unless otherwise stated, all variables identified in this subsection are specific to one intersection approach.* The bicycle is assumed to travel in the street (possibly in a bicycle lane) and in the same direction as adjacent motorized vehicles.

The methodology is focused on analyzing signalized intersection performance from the bicyclist point of view. Chapter 17, Urban Street Segments, describes a methodology for evaluating urban street performance.

The bicycle methodology is applied through a series of three steps that determine the bicycle LOS for an intersection approach. These steps are illustrated in Exhibit 18-27. Performance measures that are estimated include bicycle delay and a bicycle LOS score.

Exhibit 18-27
Bicycle Methodology for Signalized Intersections

Step 1: Determine Bicycle Delay

↓

Step 2: Determine Bicycle LOS Score for Intersection

↓

Step 3: Determine LOS

Step 1: Determine Bicycle Delay

This step describes a procedure for evaluating the performance of one intersection approach. It is repeated for each approach of interest. Bicycle delay can be calculated only for intersection approaches that have an on-street bicycle lane or a shoulder that can be used by bicyclists as a bicycle lane. Bicyclists who share a lane with automobile traffic will incur the same delay as the automobiles.

A. Compute Bicycle Lane Capacity

A wide range of capacities and saturation flow rates have been reported by many countries for bicycle lanes at intersections. Research indicates that the base saturation flow rate may be as high as 2,600 bicycles/h (*31*). However, few intersections provide base conditions for bicyclists, and current information is insufficient to calibrate a series of appropriate saturation flow adjustment factors. Until such factors are developed, it is recommended that a saturation flow rate of 2,000 bicycles/h be used as an average value achievable at most intersections.

A saturation flow rate of 2,000 bicycles/h assumes that right-turning motor vehicles yield the right-of-way to through bicyclists. Where aggressive right-turning traffic exists, 2,000 bicycles/h may not be achievable. Local observations to determine a saturation flow rate are recommended in such cases.

The capacity of the bicycle lane at a signalized intersection may be computed with Equation 18-78.

Equation 18-78

$$c_b = s_b \frac{g_b}{C}$$

where

c_b = capacity of the bicycle lane (bicycles/h),

s_b = saturation flow rate of the bicycle lane = 2,000 (bicycles/h),

g_b = effective green time for the bicycle lane (s), and

C = cycle length (s).

The effective green time for the bicycle lane can be assumed to equal that for the adjacent motor-vehicle traffic stream that is served concurrently with the subject bicycle lane (i.e., $g_b = D_p - l_1 - l_2$).

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 18-79.

Equation 18-79

$$d_b = \frac{0.5\,C\,(1 - g_b / C)^2}{1 - \min\left[\frac{v_{bic}}{c_b}, 1.0\right]\frac{g_b}{C}}$$

where d_b is bicycle delay (s/bicycle), v_{bic} is bicycle flow rate (bicycles/h), and other variables are as previously defined.

This delay equation is based on the assumption that there is no bicycle incremental delay or initial queue delay. Bicyclists will not normally tolerate an oversaturated condition and will select other routes or ignore traffic regulations to avoid the associated delays.

At most signalized intersections, the only delay to through bicycles is caused by the signal, because bicycles have the right-of-way over right-turning vehicles during the green indication. Bicycle delay could be longer than that obtained from Equation 18-79 when (*a*) bicycles are forced to weave with right-turning traffic during the green indication, or (*b*) drivers do not acknowledge the bicycle right-of-way because of high flows of right-turning vehicles.

The delay obtained from Equation 18-79 can be used to make some judgment about intersection performance. Bicyclists tend to have about the same tolerance for delay as pedestrians. They tend to become impatient when they experience a delay in excess of 30 s/bicycle. In contrast, they are very likely to comply with the signal indication if their expected delay is less than 10 s/bicycle.

Step 2: Determine Bicycle LOS Score for Intersection

This step describes a procedure for evaluating the performance of one intersection approach. It is repeated for each approach of interest. The bicycle LOS score can be calculated for any intersection approach, regardless of whether it has an on-street bicycle lane.

The bicycle LOS score for the intersection $I_{b,int}$ is calculated by using Equation 18-80 through Equation 18-83.

Equation 18-80

$$I_{b,int} = 4.1324 + F_w + F_v$$

with

Equation 18-81

$$F_w = 0.0153\,W_{cd} - 0.2144\,W_t$$

Equation 18-82

$$F_v = 0.0066\,\frac{v_{lt} + v_{th} + v_{rt}}{4\,N_{th}}$$

$$W_t = W_{ol} + W_{bl} + I_{pk} W_{os}^*$$

Equation 18-83

where

$I_{b,int}$ = bicycle LOS score for intersection;

W_{cd} = curb-to-curb width of the cross street (ft);

W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);

v_{lt} = left-turn demand flow rate (veh/h);

v_{th} = through demand flow rate (veh/h);

v_{rt} = right-turn demand flow rate (veh/h);

N_{th} = number of through lanes (shared or exclusive) (ln);

W_{ol} = width of the outside through lane (ft);

W_{bl} = width of the bicycle lane = 0.0 if bicycle lane not provided (ft);

I_{pk} = indicator variable for on-street parking occupancy = 0 if $p_{pk} > 0.0$, 1 otherwise;

p_{pk} = proportion of on-street parking occupied (decimal);

W_{os} = width of paved outside shoulder (ft); and

W_{os}^* = adjusted width of paved outside shoulder; if curb is present $W_{os}^* = W_{os} - 1.5 \geq 0.0$, otherwise $W_{os}^* = W_{os}$ (ft).

The variable "proportion of on-street parking occupied" is used to describe the presence of on-street parking and activity on the approach and departure legs of the intersection that are used by the subject bicycle movement.

Step 3: Determine LOS

This step describes a process for determining the LOS of one intersection approach. It is repeated for each approach of interest.

The bicycle LOS is determined by using the bicycle LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 18-5 to determine the LOS for the subject approach.

3. APPLICATIONS

DEFAULT VALUES

Agencies that use the methodologies in this chapter are encouraged to develop a set of local default values based on field measurements at intersections in their jurisdiction. Local default values provide the best means of ensuring accuracy in the analysis results. In the absence of local default values, the values identified in this subsection can be used if the analyst believes they are reasonable for the intersection to which they are applied.

Exhibit 18-6, Exhibit 18-7, and Exhibit 18-9 identify the input data variables associated with the automobile, pedestrian, and bicycle methodologies. These variables can be categorized as (*a*) suitable for specification as a default value or (*b*) required input data. Those variables categorized as "suitable for specification as a default value" have a minor effect on performance estimates and tend to have a relatively narrow range of typical values used in practice. In contrast, required input variables have either a notable effect on performance estimates or a wide range of possible values.

Required input variables typically represent fundamental intersection geometric elements and demand flow rates. Values for these variables should be field-measured when possible.

If field measurement of the input variables is not possible, then various options exist for determining an appropriate value for a required input variable. As a first choice, input values should be established through the use of local guidelines. If local guidelines do not address the desired variable, then some input values may be determined by considering the typical operation of (or conditions at) similar intersections in the jurisdiction. As a last option, various authoritative national guideline documents are available and should be used to make informed decisions about design options and volume estimates. The use of simple rules of thumb or "ballpark" estimates for required input values is discouraged because this use is likely to lead to a significant cumulative error in performance estimates.

Automobile Mode

The required input variables for the automobile methodology are identified in the following list. These variables represent the minimum basic input data the analyst will need to provide for an analysis and were previously defined in the text associated with Exhibit 18-6:

- Demand flow rate,
- Initial queue,
- Pedestrian flow rate,
- Bicycle flow rate,
- Number of lanes,
- Number of receiving lanes,
- Turn bay length,

- Presence of on-street parking,
- Type of signal control,
- Phase sequence,
- Left-turn operational mode,
- Speed limit, and
- Area type.

Initial queue has a significant effect on delay and can vary widely among intersections and traffic movements. If it is not possible to obtain an initial queue estimate, then the analysis period should be established so that the previous period is known to have demand less than capacity and no residual queue. A multiple-period analysis may be appropriate when the duration of congestion exceeds 15 min (i.e., 0.25 h).

Several authoritative reference documents (*32*–*34*) provide useful guidelines for selecting the type of signal control, designing the phase sequence, and selecting the left-turn operational mode (i.e., permitted, protected, or protected-permitted).

Exhibit 18-28 lists default values for the automobile methodology based on national research (*35*). Some of the values listed may also be useful for the pedestrian or bicycle methodologies. The last column of this exhibit indicates "see discussion" for some variables. In these situations, the default value is described in the discussion provided in this subsection.

Many of the controller settings are specific to an actuated phase and fully actuated signal control. If pretimed control is used and the phase durations are known, the cycle length and phase duration are set to equal the known values. If pretimed control is used and the phase durations are not known, then the quick estimation method described in Chapter 31, Signalized Intersections: Supplemental, should be used to estimate the cycle length and the duration of each phase. For semiactuated control, phases with a fixed duration should have their recall mode set to "recall-to-maximum" and their maximum green limit set to the known green interval duration.

Platoon Ratio

A default value for platoon ratio can be determined from arrival type. Once the default arrival type is determined, Exhibit 18-8 is consulted to determine the equivalent default platoon ratio for input to the methodology.

In the absence of more detailed information from Chapter 17 or field measurements, a default arrival type of 3 is used for uncoordinated through movements and a default value of 4 is used for coordinated through movements. Exhibit 18-29 provides further guidance on the relationship between arrival type, street segment length, and the provision of signal coordination for through movements.

Exhibit 18-28
Default Values: Automobile Mode with Fully or Semiactuated Signal Control

Data Category	Input Data Element	Default Values
Traffic characteristics	Right-turn-on-red flow rate	0.0 veh/h
	Percent heavy vehicles	3%
	Intersection peak hour factor	If analysis period is 0.25 h and hourly data are used: Total entering volume ≥ 1,000 veh/h: 0.92 Total entering volume < 1,000 veh/h: 0.90 Otherwise: 1.00
	Platoon ratio	See discussion
	Upstream filtering adjustment factor	1.0
	Base saturation flow rate	Metropolitan area with population ≥ 250,000: 1,900 pc/h/ln Otherwise: 1,750 pc/h/ln
	Lane utilization adjustment factor	See discussion
	On-street parking maneuver rate	See discussion
	Local bus stopping rate	When buses expected to stop Central business district: 12 buses/h Non–central business district: 2 buses/h When buses not expected to stop: 0
Geometric design	Average lane width	12 ft
	Approach grade (negative for downhill conditions)	Flat approach: 0% Moderate grade on approach: 3% Steep grade on approach: 6%
Controller settings	Dallas left-turn phasing option	Dictated by local use
	Passage time	2.0 s (presence detection)
	Maximum green	Major-street through movement: 50 s Minor-street through movement: 30 s Left-turn movement: 20 s
	Minimum green	Major-street through movement: 10 s Minor-street through movement: 8 s Left-turn movement: 6 s
	Yellow change + red clearance[a]	4.0 s
	Walk	Actuated: 7.0 s Pretimed: green interval minus pedestrian clear
	Pedestrian clear	Based on 3.5-ft/s walking speed
	Phase recall	Actuated phase: No Pretimed phase: Recall to maximum
	Dual entry	Not enabled (i.e., use single entry)
	Simultaneous gap-out	Enable
Other	Analysis period duration	0.25 h
	Stop-line detector length	40 ft (presence detection)

Note: [a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

In the absence of more detailed information from Chapter 17 or field measurements, Arrival Type 3 is used for turn movements because they are typically not coordinated.

Exhibit 18-29
Progression Quality and Arrival Type

Arrival Type	Progression Quality	Signal Spacing (ft)	Conditions Under Which Arrival Type Is Likely to Occur
1	Very poor	≤1,600	Coordinated operation on a two-way street where the subject direction does not receive good progression
2	Unfavorable	>1,600–3,200	A less extreme version of Arrival Type 1
3	Random arrivals	>3,200	Isolated signals or widely spaced coordinated signals
4	Favorable	>1,600–3,200	Coordinated operation on a two-way street where the subject direction receives good progression
5	Highly favorable	≤1,600	Coordinated operation on a two-way street where the subject direction receives good progression
6	Exceptional	≤800	Coordinated operation on a one-way street in dense networks and central business districts

Lane Utilization Adjustment Factor

The default lane utilization factors described in this subpart apply to situations in which drivers randomly choose among the exclusive-use lanes on the intersection approach. The factors do not apply to special conditions (such as short lane drops or a downstream freeway on-ramp) that might cause drivers intentionally to choose their lane position on the basis of an anticipated downstream maneuver. Exhibit 18-30 provides a summary of lane utilization adjustment factors for different lane group movements and numbers of lanes.

Exhibit 18-30
Default Lane Utilization Adjustment Factors

Lane Group Movement	Number of Lanes in Lane Group (ln)	Traffic in Most Heavily Traveled Lane (%)	Lane Utilization Adjustment Factor f_{LU}
Exclusive through	1	100.0	1.000
	2	52.5	0.952
	3[a]	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2[a]	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2[a]	56.5	0.885

Note: [a] If a lane group has more lanes than shown in this exhibit, it is recommended that field surveys be conducted or the smallest f_{LU} value shown for that type of lane group be used.

As demand approaches capacity, the analyst may use lane utilization factors that are closer to 1.0 than those offered in Exhibit 18-30. This refinement to the factor value recognizes that a high volume-to-capacity ratio is associated with a more uniform use of the available lanes because of reduced opportunity for drivers to select their lane freely.

On-Street Parking Maneuver Rate

Exhibit 18-31 gives default values for the parking maneuver rate on an intersection approach with on-street parking. It is estimated for a distance of 250 ft back from the stop line. The calculations assume 25 ft per parking space and 80% occupancy. Each turnover (one car leaving and one car arriving) generates two parking maneuvers.

Exhibit 18-31
Default Parking Maneuver Rate

Street Type	Number of Spaces in 250 ft	Parking Time Limit (h)	Turnover Rate (veh/h)	Maneuver Rate (maneuvers/h)
Two-way	10	1	1.0	16
		2	0.5	8
One-way	20	1	1.0	32
		2	0.5	16

Automobile Mode (Coordinated-Actuated Operation)

Exhibit 18-32 lists the default values for evaluating signalized intersections that are part of a coordinated-actuated signal system. The text "see discussion" in the last column of this exhibit indicates that the default value is described in the discussion provided in this part.

Exhibit 18-32
Default Values: Automobile Mode with Coordinated-Actuated Signal Control

Data Category	Input Data Element	Default Value
Controller settings	Cycle length	See discussion
	Phase splits	See discussion
	Offset	Equal to travel time in Phase 2 direction [a]
	Offset reference	End of green for Phase 2 [a]
	Force mode	Fixed

Note: [a] Assumes that Phase 2 is the reference phase. Substitute 6 if Phase 6 is the reference phase.

Cycle Length

The cycle length used for a coordinated signal system often represents a compromise value based on intersection capacity, queue size, phase sequence, segment length, speed, and progression quality. Consideration of these factors leads to the default cycle lengths shown in Exhibit 18-33.

Exhibit 18-33
Default System Cycle Length

Average Segment Length (ft)[a]	Cycle Length by Street Class and Left-Turn Phasing (s)[b]					
	Major Arterial Street			Minor Arterial Street or Grid Network		
	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets
1,300	90	120	150	60	80	120
2,600	90	120	150	100	100	120
3,900	110	120	150			

Notes: [a] Average length based on all street segments in the signal system.
[b] Selected left-turn phasing column should describe the phase sequence at the high-volume intersections in the system.

Phase Splits

If the phase splits are not known, they can be estimated by using the quick estimation method described in Chapter 31. The method can be used to estimate the effective green time for each phase on the basis of the established system cycle length. The phase split D_p is then computed by adding 4 s of lost time to the estimated effective green time (i.e., $D_p = g + 4.0$).

Nonautomobile Modes

The required input variables for the pedestrian and bicycle methodologies are identified in the following list. These variables represent the minimum basic input data the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 18-9.

- Demand flow rate of motorized vehicles,
- RTOR flow rate (pedestrian mode only),
- Permitted left-turn flow rate (pedestrian mode only),
- Pedestrian flow rate (pedestrian mode only),
- Bicycle flow rate (bicycle mode only),
- Number of lanes,
- Crosswalk length (pedestrian mode only), and
- Pedestrian signal head presence (pedestrian mode only).

The RTOR flow rate does not have a default value for application of the pedestrian methodology. This flow rate has both a notable effect on performance estimates and a wide range of possible values. The analyst is encouraged to conduct measurements at intersections for the purpose of developing local defaults for this variable.

The permitted left-turn flow rate for movements served by the permitted mode is equal to the left-turn demand flow rate. The permitted left-turn flow rate for movements served by the protected-permitted mode does not have a default value. This flow rate has both a notable effect on performance estimates and a wide range of possible values. It should be measured in the field if possible. If the analysis is dealing with future conditions or if the permitted left-turn flow rate is not known from field data, its value can be approximated as the left-turn arrival rate during the permitted period of the protected-permitted operation. This rate should equal the left-turn arrival rate during the effective red time [i.e., $q_r = (1 - P)qC/r$].

The pedestrian flow rate data consist of count data for each of five pedestrian movements at each intersection corner. These variables are shown as $v_{a,b}$, v_{ci}, v_{co}, v_{di}, and v_{do} in Exhibit 18-10.

Exhibit 18-34 lists the default values for the pedestrian and bicycle methodologies (*25–27*).

TYPES OF ANALYSIS

The automobile, pedestrian, and bicycle methodologies described in this chapter can each be used in three types (or levels) of analysis. These analysis levels are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis level are described in later parts of this subsection.

Operational Analysis

Each of the methodologies is most easily applied at an operational level of analysis. At this level, all traffic, geometric, and signalization conditions are specified as input variables by the analyst. These input variables are used in the methodology to compute various performance measures.

Exhibit 18-34
Default Values: Nonautomobile Modes

Data Category	Input Data Element	Default Value
Traffic characteristics	Intersection peak hour factor (motorized vehicles)	If analysis period is 0.25 h and hourly data are used: Total entering volume ≥ 1,000 veh/h: 0.92 Total entering volume < 1,000 veh/h: 0.90 Otherwise: 1.00
	Midsegment 85th percentile speed	Speed limit (mi/h)
	Proportion of on-street parking occupied	0.50 (if parking lane present)
Geometric design	Street width	Based on a 12-ft lane width
	Number of right-turn islands	None
	Width of outside through lane	12 ft
	Width of bicycle lane	5.0 ft (if provided)
	Width of paved outside shoulder	No parking lane: 2.0 ft (curb and gutter width) Parking lane present: 8.0 ft
	Total walkway width	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
	Crosswalk width	12 ft
	Corner radius	Trucks and buses in turn volume: 45 ft No trucks or buses in turn volume: 25 ft
Signal control	Walk	Actuated: 7 s Pretimed: green interval minus pedestrian clear
	Pedestrian clear	Based on 3.5-ft/s walking speed
	Rest in walk	Not enabled
	Cycle length	Based on default values determined for automobile mode
	Yellow change + red clearance [a]	4 s
	Duration of phases serving pedestrians and bicycles	Based on default values determined for automobile mode
Other	Analysis period duration	0.25 h

Note: [a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

Design Analysis

The design level of analysis has two variations. Both variations require specifying (*a*) traffic conditions and (*b*) target levels for a specified set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

The second variation of the design level requires the additional specification of geometric conditions. The methodology is then applied by using an iterative approach in which alternative signalization conditions are separately evaluated.

The objective with either variation is to identify alternatives that operate at the target level of the specified performance measures (or provide a better level of performance). The analyst may then recommend the “best” alternative based on consideration of the full range of factors.

Planning and Preliminary Engineering Analysis

The planning and preliminary engineering level of analysis is intended to provide an estimate of the LOS for either a proposed intersection or an existing intersection in a future year. This level of analysis may also be used for a preliminary engineering activity to size the overall geometrics of a proposed intersection.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose were described previously in this section.

The requirement for a complete description of the signal timing plan can be a burden for some planning analyses, especially when the signal control is pretimed or coordinated-actuated. The quick estimation method described in Chapter 31 can be used to estimate a reasonable timing plan, in conjunction with the aforementioned default values.

USE OF ALTERNATIVE TOOLS

General alternative tool guidance is provided in Chapters 6 and 7.

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for applying alternative tools to the analysis of signalized intersections. Additional information on this topic may be found in the Technical Reference Library in Volume 4.

Strengths of the Automobile Methodology

The automobile methodology described in Section 2 offers a comprehensive procedure for analyzing the performance of a signalized intersection. It models the driver–vehicle–road–signal system with reasonable accuracy for most applications. Simulation-based traffic analysis tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with the roadway and the control system. As such, some simulation tools can model the driver–vehicle–road–signal system more accurately for some applications.

The automobile methodology offers the following advantages over the use of simulation-based analysis tools:

- Its empirically calibrated saturation flow rate adjustment factors can produce an accurate estimate of saturation flow rate (simulation tools require saturation flow rate as an input variable).
- It produces a direct estimate of capacity and volume-to-capacity ratio (these measures are much more difficult to quantify with simulation).
- It produces an estimate of expected, long-run performance for a variety of measures (multiple runs and supplemental calculations are required to obtain this type of estimate with a simulation tool).

Identified Limitations of the Automobile Methodology

The limitations of the automobile methodology are identified near the end of Section 1. If any of these limitations applies to a particular situation, then alternative tools may produce more credible performance estimates. Limitations involving consideration of the impact of progression on performance are a special case that is discussed in more detail in Chapter 16, Urban Street Facilities.

Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the capacity, control delay, LOS, and back of queue associated with a lane group at a signalized intersection. Alternative tools often offer additional performance measures such as number of stops, fuel consumption, air quality, and operating costs.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS assessment for signalized intersections is based on control delay, which is defined as the excess travel time caused by the action of the control device (in this case, the signal).

Simulation-based analysis tools often use a definition of delay that is different from that used in the automobile methodology, especially for movements that are oversaturated at some point during the analysis. Therefore, some care must be taken in the determination of LOS when simulation-based delay estimates are used. Delay comparison among different tools is discussed in more detail in Chapter 7.

An accurate estimate of control delay may be obtained from a simulation tool by performing simulation runs with and without the control device(s) in place. The segment delay reported with no control is the delay due to geometrics and interaction between vehicles. The additional delay reported in the run with the control in place is, by definition, the control delay.

Conceptual Differences That Preclude Direct Comparison of Results

Conceptual differences in modeling approach may preclude the direct comparison of performance measures from the automobile methodology with those from alternative tools. The treatment of random arrivals is a case in point. There is a common misconception among analysts that alternative tools treat random arrivals in a similar manner.

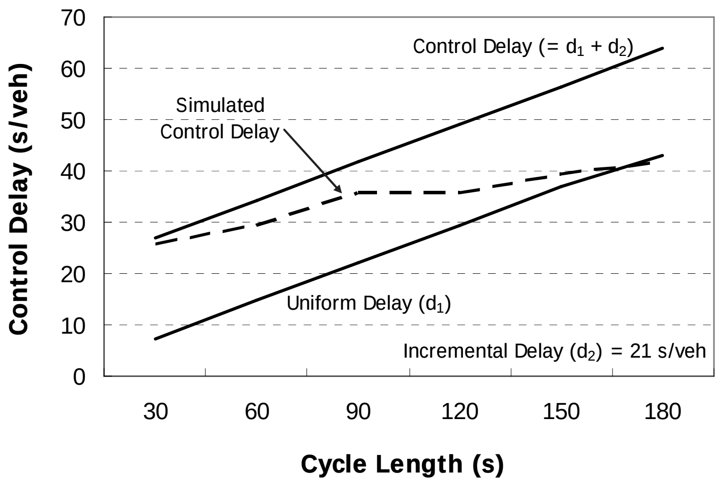
A simple case is used to demonstrate the different ways alternative tools model random arrivals. Consider an isolated intersection with a two-phase sequence. The subject intersection approach serves only a through movement; there are no turning movements from upstream intersections or driveways. The only parameter that is allowed to vary in this example is the cycle length (all other variables are held constant).

The results of this experiment are shown in Exhibit 18-35. The two solid lines represent delay estimates obtained from the automobile methodology. Uniform delay is shown to increase linearly with cycle length. Incremental delay is constant with respect to cycle length because the volume-to-capacity ratio is

constant. As a result, control delay (being the sum of the uniform and incremental delay) is also shown to increase linearly with cycle length.

Exhibit 18-35
Effect of Cycle Length on Delay

LIVE GRAPH
Click here to view



The dashed line represents the control delay estimate obtained from a simulation-based analysis tool. The simulation-based tool shows close agreement with the automobile methodology for short cycles but deviates for longer cycles. There are likely to be explainable reasons for this difference; however, the point is that such differences are likely to exist among tools. The analyst should understand the underlying modeling assumptions and limitations inherent in any tool (including the automobile methodology) when it is used. Moreover, the analyst should fully understand the definition of any performance measure used so as to interpret the results and observed trends properly.

Adjustment of Alternative Tool Parameters

For applications in which either an alternative tool or the automobile methodology can be used, some adjustment is generally required for the alternative tool if some consistency with the automobile methodology is desired. For example, the parameters that determine the capacity of a signalized approach (e.g., saturation flow rate and start-up lost time) should be adjusted to ensure that the simulated lane group (or approach) capacities match those estimated by the automobile methodology.

Step-by-Step Recommendations for Applying Alternative Tools

This part provides recommendations specifically for signalized intersection evaluation. The following steps should be taken to apply an alternative tool for signalized intersection analysis:

1. Determine whether the automobile methodology can provide a realistic assessment of the capacity and control delay for the signalized approaches of interest. The limitations stated at the end of Section 1 provide a good starting point for this assessment. If there are no conditions outside these limitations, then it should not be necessary to consider alternative tools. Otherwise, proceed with the remaining steps.

2. Select the appropriate tool in accordance with the general guidelines presented in Chapter 7.
3. Enter all available input characteristics and parameters.
4. Use the tool to evaluate the intersection. Be careful to observe the guidance provided in Chapter 7 regarding self-aggravating conditions that occur near capacity. If the tool is simulation based, then estimate the required number of runs so that the comparison is statistically valid.
5. If the documented delay definition and computational methodology used by the tool conform to the specifications set forth in Chapter 7 of this manual, then the delay estimates should be suitable for estimating the LOS. Otherwise, no such estimate should be attempted.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 31 includes example problems that address the following conditions:

- Left-turn storage bay overflow,
- RTOR operation,
- Short through lanes, and
- Closely spaced intersections.

4. EXAMPLE PROBLEMS

INTRODUCTION

This part of the chapter describes the application of each of the automobile, pedestrian, and bicycle methodologies through the use of example problems. Exhibit 18-36 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operational analysis level in terms of the calculations, except that default values are used when field-measured values are not available.

Exhibit 18-36
Example Problems

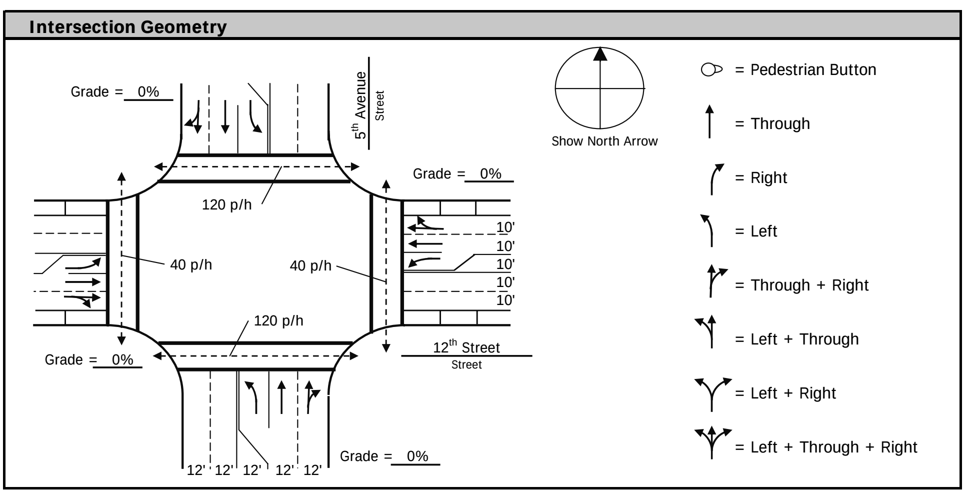
Problem Number	Description	Analysis Level
1	Automobile LOS	Operational
2	Pedestrian LOS	Operational
3	Bicycle LOS	Operational

EXAMPLE PROBLEM 1: AUTOMOBILE LOS

The Intersection

The intersection of 5th Avenue and 12th Street is an intersection of two urban arterial streets. It is shown in Exhibit 18-37.

Exhibit 18-37
Example Problem 1: Intersection Plan View



The Question

What is the motorist delay and LOS during the analysis period for each lane group and the intersection as a whole?

The Facts

The intersection's traffic, geometric, and signalization conditions are listed in Exhibit 18-38 and Exhibit 18-39.

Exhibit 18-38
Example Problem 1: Signal Conditions

Controller Data Worksheet

General Information

Analyst:	BR	Intersection:	5th Avenue/12th Street
Agency or Company:		Area Type: CBD	Phase 2: EB
Date Performed:	2/11/2010		
Analysis Time Period:	5:30 pm to 5:45 pm	Analysis Year:	2010
Filename:	C:\Documents and Settings\TexasEX3		

Phase Sequence and Left-Turn Mode

WB left (1) with WB thru (6)	EB left (5) with EB thru (2)	NB left (3) before SB thru (4)	SB left (7) before NB thru (8)
WB left permitted	EB left permitted	NB left (3) prot-perm	SB left (7) prot-perm

Phase Settings

Approach	Eastbound		Westbound		Northbound		Southbound	
Phase number		2		6	3	8	7	4
Movement		L+T+R		L+T+R	L	T+R	L	T+R
Lead/lag left-turn phase		--		--	Lead	--	Lead	--
Left-turn mode		Perm.		Perm.	Pr/Pm	--	Pr/Pm	--
Passage time, s		2.0		2.0	2.0	2.0	2.0	2.0
Maximum green, s		30		30	25	50	25	50
Minimum green, s		5		5	5	5	5	5
Yellow change, s		4.0		4.0	4.0	4.0	4.0	4.0
Red clearance, s		0.0		0.0	0.0	0.0	0.0	0.0
Walk+ ped. clear, s		19		19		21		21
Recall?:	No	No	No	No	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes

Enable Simultaneous Gap-Out (check = Yes)? Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑	Enable Dallas Left-Turn Phasing? Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐

Protected right-turn with left-turn phase?	n.a.		n.a.		Eastbd. right		Westbd. right	
	No		No		No		No	

Phase number assignment to timers (by ring):					Controller timer ring structure:			
Ring 1:	0	2	3	4	Ring 1:	Timer 1	Timer 2	Timer 3	Timer 4
Ring 2:	0	6	7	8	Ring 2:	Timer 5	Timer 6	Timer 7	Timer 8

Exhibit 18-39
Example Problem 1: Traffic and Geometric Conditions

Movement-Specific Intersection Data Worksheet

Approach	Eastbound			Westbound			Northbound			Southbound		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Traffic Characteristics *(Enter the volume data in all columns. For all other blue cells, enter values only if there are one or more lanes.)*												
Volume, veh/h	71	318	106	118	600	24	133	1,644	111	194	933	111
Right-turn-on-red volume, veh/h			0			0			22			33
Percent heavy vehicles, %	5	5		5	5		2	2		2	2	
Lane utilization adjustment factor	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Peak hour factor 1.00	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Start-up lost time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Extension of eff. green time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Platoon ratio	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Upstream filtering factor	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Pedestrian volume, p/h		120			120			40			40	
Bicycle volume, bicycles/h		0			0			0			0	
(future use)												
Initial queue, veh	0	0		0	0		0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
(future use)												
Multiple-Period Analysis Counts *(if all cell values = 0, then values in the 'Volume' row above will be used for a single-period analysis)*												
Period **1** traffic count, veh												
Period **2** traffic count, veh												
Period **3** traffic count, veh												
Period **4** traffic count, veh												
Intersection Approach Characteristics *(Enter the number of lanes. For all blue cells, enter values only if there are one or more lanes.)*												
Number of lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Average lane width, ft	10.0	10.0		10.0	10.0		12.0	12.0		12.0	12.0	
Number of receiving lanes		2			2			2			2	
Turn bay or segment length, ft	200	999		200	999		200	999		200	999	
Approach Data	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*
Parking present?	No		Yes	No		Yes	No		No	No		No
Parking maneuvers, maneuvers/h	0		5	0		5	0		0	0		0
Bus stopping rate, buses/h			0			0			0			0
Approach grade, %	0	0	0	0	0	0	0	0	0	0	0	0
Detection Data *(Enter values only if there are one or more lanes.)*												
Stop line detector length, ft	40	40	n.a.	40	40	n.a.	40	40	n.a.	40	40	n.a.
(future use)												

The intersection is located in a central business district–type environment. Adjacent signals are somewhat distant so the intersection is operated by using fully actuated control. Vehicle arrivals to each approach are characterized as "random" and are described by using a platoon ratio of 1.0.

The left-turn movements on the north–south street operate under protected-permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the east–west street operate as permitted.

All intersection approaches have a 200-ft left-turn bay, an exclusive through lane, and a shared through and right-turn lane. The average width of the traffic lanes on the east–west street is 10 ft. The average width of the traffic lanes on the north–south street is 12 ft.

Crosswalks are provided on each intersection leg. A two-way flow rate of 120 p/h is estimated to use each of the east–west crosswalks and a two-way flow rate of 40 p/h is estimated to use each of the north–south crosswalks.

On-street parking is present on the east–west street. It is estimated that parking maneuvers on each intersection approach occur at a rate of 5 maneuvers/h during the analysis period.

The speed limit is 35 mi/h on each intersection approach. The analysis period is 0.25 h. There is no initial queue for any movement.

As noted in the next section, none of the lane groups at the intersection has two or more exclusive lanes. For this reason, the saturation flow rate adjustment factor for lane utilization is equal to 1.0 for all approaches. Any unequal lane use that may occur due to the shared through and right-turn lane groups will be accounted for in the lane group flow rate calculation, as described in Chapter 31.

Outline of Solution

Movement-Based Data

Exhibit 18-40 provides a summary of the analysis of the individual traffic movements at the intersection. The movement numbers shown follow the numbering convention in Exhibit 18-2.

Exhibit 18-40
Example Problem 1: Movement-Based Output Data

Movement:	EB L 5	EB T 2	EB R 12	WB L 1	WB T 6	WB R 16	NB L 3	NB T 8	NB R 18	SB L 7	SB T 4	SB R 14
Volume, veh/h	71	318	106	118	600	24	133	1,644	89	194	933	78
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj. Factor (A_pbT)	0.999		0.878	0.976		0.878	0.999		0.976	1.000		0.977
Parking, Bus Adj. Factors (f_bb x f_p)	1.000	1.000	0.875	1.000	1.000	0.875	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted Sat. Flow Rate, veh/h/ln	1,629	1,629	1,629	1,629	1,629	1,629	1,676	1,676	1,676	1,676	1,676	1,676
Lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane Assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Capacity, veh/h	147.23	629.27	201.44	205.81	853.60	34.08	326.46	1,545.51	83.10	224.96	1,604.59	134.14
Proportion Arriving On Green	0.294	0.294	0.294	0.294	0.294	0.294	0.061	0.491	0.491	0.097	0.527	0.527
Approach Volume, veh/h		495			742			1,866			1,205	
Approach Delay, s/veh		32.553			37.432			71.532			19.828	

Note: n.a. = not applicable

Two saturation flow rate adjustment factors are shown in Exhibit 18-40. One factor is the pedestrian–bicycle adjustment factor. This factor is used to estimate the saturation flow rate for the turn movement in a lane group. The "parking, bus adjustment factor" represents the product of the parking adjustment factor and the bus blockage adjustment factor. This combined factor is computed separately for the lane group that is adjacent to the parking or bus stop.

The adjusted saturation flow rate represents the saturation flow rate for all lane groups on the approach. It reflects the combined effect of lane width, heavy-vehicle presence, grade, and area type. The effect of pedestrians, bicycles, parking, bus blockage, lane utilization, right-turn maneuvers, and left-turn

maneuvers is calculated separately at a later stage of the analysis because their values are influenced by signal timing, lane group demand flow rate, and lane group location (adjacent to parking or not, etc.). As such, these factors are internal to the iterative sequence of calculations used to estimate signal phase duration.

Capacity for a movement is computed by using the movement volume proportion in the lane group, lane group saturation flow rate, and corresponding phase duration. This variable represents the capacity of the movement, regardless of whether it is served in an exclusive lane or in a shared lane. If the movement is served in a shared lane, then the movement capacity represents the portion of the lane group capacity available to the movement, as distributed in proportion to the flow rate of the movements served by the associated lane group.

The last two rows in Exhibit 18-40 represent summary statistics for the approach. The approach volume represents the sum of the three movement volumes. Approach delay is computed as volume-weighted average for the lane groups served on an intersection approach.

Timer-Based Phase Data

Exhibit 18-41 provides a summary of the output data by using a signal controller perspective. The controller has eight timing functions (or timers), with Timers 1 to 4 representing Ring 1 and Timers 5 to 8 representing Ring 2. The ring structure and phase assignments were previously shown at the bottom of Exhibit 18-38. Timers 1 and 5 are not used at this intersection.

Exhibit 18-41
Example Problem 1: Timer-Based Phase Output Data

Timer Data

Timer:	1	2 EB L.T.T+R	3 NB L	4 SB T.T+R	5	6 WB L.T.T+R	7 SB L	8 NB T.T+R
Assigned Phase		2	3	4		6	7	8
Case No		6	1	4		6	1	4
Phase Duration (G+Y+Rc), s		34.00	10.21	57.66		34.00	13.87	54.00
Change Period (Y+Rc), s		4.00	4.00	4.00		4.00	4.00	4.00
Max. Allowable Headway (MAH), s		3.44	3.13	3.06		3.44	3.13	3.06
Maximum Green Setting (Gmax), s		30.00	25.00	50.00		30.00	25.00	50.00
Max. Queue Clearance Time (g_c+I1), s		31.10	6.16	23.29		29.51	9.61	52.00
Green Extension Time (g_e), s		0.000	0.199	7.831		0.238	0.296	0.000
Probability of Phase Call (p_c)		1.000	0.977	1.000		1.000	0.996	1.000
Probability of Max Out (p_x)		1.000	0.000	0.179		1.000	0.000	1.000

Equilibrium Cycle Length, s: 102

The timing function construct is essential in modeling a ring-based signal controller. *Timers* always occur in the same numeric sequence (i.e., 1 then 2 then 3 then 4 in Ring 1; 5 then 6 then 7 then 8 in Ring 2). The practice of associating movements to phases (e.g., the major-street through movement to Phase 2) coupled with the occasional need for lagging left-turn phases and split phasing creates the situation in which *phases* do not always time in sequence. For example, with a lagging left-turn phase sequence, major-street through Phase 2 times first and then major-street left-turn Phase 1 times second.

The modern controller accommodates the assignment of phases to timing functions by allowing the ring structure to be redefined manually or by time-of-day settings. Specification of this structure is automated in the computational engine by assigning phases to timers.

The methodology is based on modeling *timers*, not by directly modeling movements or phases. The methodology converts movement and phase input data into timer input data. It then models controller response to these inputs and computes timer duration and related performance measures.

The signalized intersection in this example problem has a lead–lead left-turn phase sequence on the north–south street. Hence, the timer numbers for this street are the same as the phase numbers, which are the same as the movement numbers (e.g., the northbound left-turn Movement 3 is associated with Phase 3, which is assigned to Timer 3). In contrast, the east–west street does not have left-turn phases, so one timer and one phase are used to serve all movements on a given approach.

The case number shown in Exhibit 18-41 is used as a single variable descriptor of each possible combination of left-turn mode and lane-group type (i.e., shared or exclusive). An understanding of this variable is not needed to interpret the output data.

The phase duration shown in the exhibit represents the estimated average phase duration during the analysis period. It represents the sum of the green, yellow change, and red clearance intervals. For Timer 2 (i.e., Phase 2), the average green interval duration is 30 s (= 34.00 – 4.00).

The durations of Phases 2, 3, and 4 add to the average cycle length of 101.87 s (= 34.00 + 10.21 + 57.66). Similarly, the durations of Phases 6, 7, and 8 add to the cycle length.

The cycle length is described in Exhibit 18-41 to be the "equilibrium" cycle length. The equilibrium cycle length is the average cycle length when all phase durations are dictated by traffic demand. However, the duration of several phases at this intersection is constrained by their maximum green limit. As such, the cycle length shown is not truly an equilibrium cycle length for this particular intersection.

The maximum green setting is input by the analyst. If the intersection were operated as coordinated-actuated, the "equivalent" maximum green setting would be shown here. It would be computed from the input phase splits and would reflect the specified force mode.

The maximum queue clearance time represents the largest queue clearance time of all lane groups served by the phase. Queue clearance time represents the time between the start of the green interval and the end of the queue service period. It is determined from the queue accumulation polygon. It includes the start-up lost time.

The maximum allowable headway, maximum green, and maximum queue clearance time apply only to actuated phases. They are not relevant to calculation of coordinated phase duration.

The green extension time represents the time the green interval is extended by arriving vehicles. This value is 0.0 s for two timers because they terminate by extension to their maximum limit (i.e., max-out).

The probability of a phase call represents the probability that one or more vehicles will place a call for service on the associated timer. The probability of

max-out represents the probability that the phase will extend to the maximum green setting and terminate, perhaps leaving some unserved vehicles on the intersection approach.

Timer-Based Movement Data

Exhibit 18-42 summarizes the output for the vehicle movements assigned to each timer. Separate sections of output are shown in the exhibit for the left-turn, through, and right-turn movements. The assigned movement row identifies the movement (previously identified in Exhibit 18-40) assigned to each timer.

Exhibit 18-42
Example Problem 1: Timer-Based Movement Output Data

Timer Data								
Timer:	**1**	**2 EB L.T.T+R**	**3 NB L**	**4 SB T.T+R**	**5**	**6 WB L.T.T+R**	**7 SB L**	**8 NB T.T+R**
Left-Turn Movement Data								
Assigned Movement		5	3			1	7	
Mvmt. Sat Flow, veh/h		696.73	1,592.65			818.40	1,592.65	
Through Movement Data								
Assigned Movement		2		4		6		8
Mvmt. Sat Flow, veh/h		2,136.77		3,046.34		2,898.49		3,148.76
Right-Turn Movement Data								
Assigned Movement		12		14		16		18
Mvmt. Sat Flow, veh/h		684.02		254.67		115.71		169.31

The saturation flow rate shown in Exhibit 18-42 represents the saturation flow rate computed for the movement. For through movements in exclusive lanes, the movement saturation flow rate is equal to the number of through lanes times the adjusted saturation flow rate, times the pedestrian–bicycle adjustment factor, times the combined parking–bus blockage adjustment factor. For turn movements in exclusive lanes, the calculation is similar except that the left-turn (or right-turn) adjustment factor is also applied.

For turn movements that share a lane with a through movement, the saturation flow rate for the lane group is computed by using the procedure described in Chapter 31. The movement saturation flow rate represents the portion of the lane group saturation flow rate available to the movement, as distributed in proportion to the flow rate of the movements served by the lane group. To illustrate this point, consider Timer 4. It has a shared-lane lane group with 15.7% right-turning vehicles, 84.3% through vehicles, and a saturation flow rate of 1,624.5 veh/h/ln. The turn movement saturation flow rate is 254.67 veh/h (= 0.157 × 1,624.5). The through movement saturation flow rate in this shared lane is 1,369.8 veh/h (= 0.843 × 1,624.5). The through movement is also served by one exclusive through lane with a saturation flow rate of 1,676.5 veh/h. Thus, the total through-movement saturation flow rate is 3,046.3 veh/h (= 1,369.8 + 1,676.5). The individual lane group saturation flow rates used in this example were obtained from the lane group data described in the next few sections.

Timer-Based Left Lane Group Data

Exhibit 18-43 summarizes the output for the "left" lane group associated with an intersection approach. Each left lane group includes the left-turn movements when they exist on an intersection approach. A left lane group will also contain all the output data for a single-lane approach, regardless of whether a left-turn movement exists.

The "lane assignment" row indicates the lane groups served by the timer (e.g., L, left turn; T, through; R, right turn). The letter "L" is shown for Timers 2 and 6 as a reminder that the timer is serving a left-turn lane group. Other letter combinations are possible. For example, "L+T" indicates the timer is serving a lane group consisting of a shared lane serving left-turn and through movements. A "L+T+R" sequence indicates a single-lane approach serving all movements.

Exhibit 18-43
Example Problem 1: Timer-Based Left Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Left Lane Group Data								
Assigned Movement		5	3			1	7	
Lane Assignment		L	L (Pr/Pm)			L	L (Pr/Pm)	
Lanes in Group		1	1			1	1	
Group Volume (v), veh/h		71.0	133.0			118.0	194.0	
Group Sat. Flow (s), veh/h/ln		696.7	1,592.6			818.4	1,592.6	
Queue Serve Time (g_s), s		10.289	4.160			14.328	7.613	
Cycle Queue Clear Time (g_c), s		29.097	4.160			27.508	7.613	
*Perm LT Sat Flow Rate (s_l), veh/h/l		696.7	499.3			818.4	250.4	
*Shared LT Sat Flow (s_sh), veh/h/ln		0.0	0.0			0.0	0.0	
*Perm LT Eff. Green (g_p), s		30.00	50.00			30.00	55.31	
*Perm LT Serve Time (g_u), s		11.19	32.37			16.82	0.00	
*Perm LT Que Serve Time (g_ps), s		10.29	6.40			14.33	0.00	
*Time to First Blk (g_f), s		0.00	0.00			0.00	0.00	
*Serve Time pre Blk (g_fs), s		0.00	0.00			0.00	0.00	
*Proportion LT Inside Lane (P_L)		1.000	1.000			1.000	1.000	
Lane Group Capacity (c), veh/h		147.2	326.5			205.8	225.0	
Volume-to-Capacity Ratio (X)		0.482	0.407			0.573	0.862	
Available Capacity (c_a), veh/h		147.2	620.2			205.8	461.5	
Upstream Filter Factor (I)		1.000	1.000			1.000	1.000	
Uniform Delay (d1), s/veh		44.936	13.243			41.483	30.229	
Incremental Delay (d2), s/veh		0.910	0.304			2.496	3.791	
Initial Queue Delay (d3), s/veh		0.000	0.000			0.000	0.000	
Control Delay (d), s/veh		45.846	13.547			43.979	34.020	
First-Term Queue (Q1), veh/ln		1.75	1.39			2.83	2.98	
Second-Term Queue (Q2), veh/ln		0.04	0.03			0.14	0.24	
Third-Term Queue (Q3), veh/ln		0.00	0.00			0.00	0.00	
Percentile bk-of-que factor (f_B%)		1.00	1.00			1.00	1.00	
Percentile Back of Queue (Q%), veh/ln		1.78	1.42			2.97	3.22	
Percentile Storage Ratio (RQ%)		0.232	0.180			0.386	0.409	
Initial Queue (Qb), veh		0.0	0.0			0.0	0.0	
Final (Residual) Queue (Qe), veh		0.0	0.0			0.0	0.0	
Saturated Delay (ds), s/veh		0.000	0.000			0.000	0.000	
Saturated Queue (Qs), veh		0.00	0.00			0.00	0.00	
Saturated Capacity (cs), veh/h		0.0	0.0			0.0	0.0	
Initial Queue Clear Time (tc), h		0.000	0.000			0.000	0.000	

The lane assignment row also indicates the operational mode for the left-turn movements. "Prot" indicates a protected left-turn mode. "Pr/Pm" indicates a protected-permitted left-turn mode. Other designations with the letter "L" indicate either a permitted left-turn mode or split phasing.

The rows listed in Exhibit 18-43 that start with "queue serve time" and end with "uniform delay" correspond to variables that are computed from the queue accumulation polygon.

The permitted left-turn saturation flow rate represents the filtering flow rate of a permitted left-turn movement. Equations for computing this flow rate and the other variables identified with an asterisk (*) are described in Chapter 31.

The shared left-turn saturation flow rate is the saturation flow rate of a shared left-turn and through lane during the period after the first blocking left-turning vehicle arrives but before the queue service ends. This flow rate is applicable only when the opposing approach has one traffic lane. It reflects the opportunities to serve the subject approach that are created by left-turning vehicles in the opposing lane.

The permitted left-turn effective green time represents the time available for permitted left-turn movement. In general, it is the time in the opposing through movement phase that is associated with a permissive green ball signal indication. Its duration can vary with phase sequence and timing.

The permitted left-turn service time represents the time required to serve the left-turn queue. This time occurs during the permitted left-turn effective green time but after the conflicting queue clears. It exists for phases that operate in the permitted mode or in the protected-permitted mode.

The time to first block applies to a lane group with a shared lane and a left-turn movement that operates in the permitted or protected-permitted mode. It represents the time from the start of the through phase until the first left-turning vehicle arrives at the stop line and stops to wait for an acceptable gap in oncoming traffic.

The queue service time before the first block (i.e., serve time pre blk) represents the queue service time for a stream of through movements in a shared left-turn and through lane. If the left-turn flow rate is low, the time to first block may occur well into the phase. In this case, it is possible that the queue of through vehicles in the shared lane will be served before the first left-turning vehicle arrives. This variable applies only to lane groups with a shared left-turn lane.

The proportion of left-turning vehicles in the inside lane represents the distribution of vehicles in the left-lane group. If a left-turn bay exists, then the proportion equals 1.0. If the lane group is shared by left-turn and through movements, then the proportion can vary between 0.0 and 1.0. If it is 1.0, then the shared lane operates as an exclusive left-turn lane.

Uniform delay represents the area under the queue accumulation polygon. This polygon is based on an average arrival rate during the green indication and an average arrival rate during the red indication. As such, it reflects the effect of progression on the delay estimate.

The available capacity is computed for all actuated phases and noncoordinated phases. It is computed by using the maximum green setting for the phase. For coordinated phases, the available capacity is computed by using the average effective green time.

The incremental delay is computed by using the incremental delay equation. For actuated phases, it uses available capacity to estimate the incremental delay factor k. For coordinated phases and phases set to "recall-to-maximum," it uses a factor of 0.50.

The first-term queue is a back-of-queue estimate that is obtained from an arrival–departure polygon. This polygon is based on the specification of arrival rates during the red and green intervals. As such, it reflects the effect of progression on first-term queue size. The procedure for developing this polygon is described in Chapter 31.

The second-term queue is computed as a derivative of the incremental delay estimate. It represents the average number of vehicles in queue each cycle due to

random variation in arrivals plus those vehicles in queue due to oversaturation during the analysis period.

The queue storage ratio represents the ratio of the back-of-queue size to the available storage length. In general, this ratio can be computed for turn bays and through lanes; however, it is computed only for the left-turn bays in this example. A value of 0.0 indicates that no turn vehicles are queued in the bay. A value of 1.0 or more indicates that the queue completely fills the bay at some point during the cycle.

The initial queue reflects the input initial queue value when a single analysis period is evaluated. In contrast, it reflects the residual queue from the previous analysis period for the second and subsequent analysis periods of a multiple-period analysis.

The saturated delay, queue, and capacity data reflect the output from a complete (and separately computed) intersection analysis. For this separate analysis, lane groups with an initial queue will have their demand flow rate adjusted so that volume equals lane group capacity. The saturated delay equals the uniform delay computed for this "saturated" condition. Similarly, the "saturated" queue equals the first-term queue for the saturated condition.

The initial queue clear time indicates the time when the last vehicle that arrives at an overflow queue during the analysis period clears the intersection (measured from the start of the analysis period).

Timer-Based Middle Lane Group Data

Exhibit 18-44 provides a summary of the output for the "middle" lane group associated with an intersection approach. This lane group is used when one or more exclusive lanes serve through vehicles on an intersection approach. The explanation of the various output statistics is the same as that previously given for the left lane groups.

In Exhibit 18-44, the exclusive through lane served by Timer 8 has a volume-to-capacity ratio that slightly exceeds 1.0. This condition results in a large value of control delay (= 73.6 s/veh) and a final (i.e., residual) queue size of 11.8 veh. The last vehicle to arrive at this queue during the analysis period will depart the intersection 0.264 h after the *start* of the 0.25-h analysis period.

Timer-Based Right Lane Group Data

Exhibit 18-45 summarizes the output for the "right" lane group associated with an intersection approach. This lane group is used when there are two or more lanes on an intersection approach and a through or right-turn movement is present. A lane that is shared by the right-turn and through movements is always shown in the right lane group. The explanation of the various output statistics is the same as that previously given for the left lane groups.

The protected right-turn saturation flow rate row is used when the right-turn movement is provided a green arrow indication concurrently with its complementary left-turn phase on the cross street. This flow rate represents the saturation flow rate during the green arrow. Similarly, the protected right-turn effective green time equals the effective green time coincident with the green

arrow indication. This operation is not provided at the subject intersection, so the values for these two variables equal 0.0.

Exhibit 18-44
Example Problem 1: Timer-Based Middle Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Middle Lane Group Data								
Assigned Movement		2		4		6		8
Lane Assignment		T		T		T		T
Lanes in Group		1		1		1		1
Group Volume (v), veh/h		239.2		513.4		336.6		870.1
Group Sat. Flow (s), veh/h/ln		1,628.6		1,676.5		1,628.6		1,676.5
Queue Serve Time (g_s), s		12.376		21.284		18.724		50.000
Cycle Queue Clear Time (g_c), s		12.376		21.284		18.724		50.000
Lane Group Capacity (c), veh/h		479.6		883.0		479.6		822.9
Volume-to-Capacity Ratio (X)		0.499		0.581		0.702		1.057
Available Capacity (c_a), veh/h		479.6		883.0		479.6		822.9
Upstream Filter Factor (I)		1.000		1.000		1.000		1.000
Uniform Delay (d1), s/veh		29.717		16.445		31.956		25.934
Incremental Delay (d2), s/veh		0.299		0.649		3.876		47.658
Initial Queue Delay (d3), s/veh		0.000		0.000		0.000		0.000
Control Delay (d), s/veh		30.017		17.094		35.832		73.592
First-Term Queue (Q1), veh/ln		4.73		7.61		7.15		18.26
Second-Term Queue (Q2), veh/ln		0.04		0.16		0.52		10.89
Third-Term Queue (Q3), veh/ln		0.00		0.00		0.00		0.00
Percentile bk-of-que factor (f_B%)		1.00		1.00		1.00		1.00
Percentile Back of Queue (Q%), veh/ln		4.77		7.77		7.67		29.15
Percentile Storage Ratio (RQ%)		0.124		0.198		0.200		0.741
Initial Queue (Qb), veh		0.0		0.0		0.0		0.0
Final (Residual) Queue (Qe), veh		0.0		0.0		0.0		11.8
Saturated Delay (ds), s/veh		0.000		0.000		0.000		0.000
Saturated Queue (Qs), veh		0.00		0.00		0.00		0.00
Saturated Capacity (cs), veh/h		0.0		0.0		0.0		0.0
Initial Queue Clear Time (tc), h		0.000		0.000		0.000		0.264

Exhibit 18-45
Example Problem 1: Timer-Based Right Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Right Lane Group Data								
Assigned Movement		12		14		16		18
Lane Assignment		T+R		T+R		T+R		T+R
Lanes in Group		1		1		1		1
Group Volume (v), veh/h		184.8		497.6		287.4		862.9
Group Sat. Flow (s), veh/h/ln		1,192.2		1,624.5		1,385.6		1,641.6
Queue Serve Time (g_s), s		13.179		21.285		18.808		50.000
Cycle Queue Clear Time (g_c), s		13.179		21.285		18.808		50.000
*Prot RT Sat Flow Rate (s_R), veh/h/ln		0.000		0.000		0.000		0.000
*Prot RT Eff. Green (g_R), s		0.000		0.000		0.000		0.000
*Proportion RT Outside Lane (P_R)		0.574		0.157		0.084		0.103
Lane Group Capacity (c), veh/h		351.1		855.7		408.1		805.7
Volume-to-Capacity Ratio (X)		0.526		0.581		0.704		1.071
Available Capacity (c_a), veh/h		351.1		855.7		408.1		805.7
Upstream Filter Factor (I)		1.000		1.000		1.000		1.000
Uniform Delay (d1), s/veh		30.001		16.445		31.986		25.934
Incremental Delay (d2), s/veh		0.729		0.670		4.631		52.458
Initial Queue Delay (d3), s/veh		0.000		0.000		0.000		0.000
Control Delay (d), s/veh		30.729		17.116		36.617		78.392
First-Term Queue (Q1), veh/ln		3.68		7.37		6.11		17.88
Second-Term Queue (Q2), veh/ln		0.07		0.16		0.52		11.74
Third-Term Queue (Q3), veh/ln		0.00		0.00		0.00		0.00
Percentile bk-of-que factor (f_B%)		1.00		1.00		1.00		1.00
Percentile Back of Queue (Q%), veh/ln		3.76		7.53		6.64		29.62
Percentile Storage Ratio (RQ%)		0.098		0.192		0.173		0.753
Initial Queue (Qb), veh		0.0		0.0		0.0		0.0
Final (Residual) Queue (Qe), veh		0.0		0.0		0.0		14.3
Saturated Delay (ds), s/veh		0.000		0.000		0.000		0.000
Saturated Queue (Qs), veh		0.00		0.00		0.00		0.00
Saturated Capacity (cs), veh/h		0.0		0.0		0.0		0.0
Initial Queue Clear Time (tc), h		0.000		0.000		0.000		0.268

Results

A comparison of the lane-group volumes in Exhibit 18-43, Exhibit 18-44, and Exhibit 18-45 indicates the extent to which drivers are expected to distribute themselves among the lane groups on each intersection approach. For example, Timer 2 serves three lane groups on the eastbound approach. The left lane group is an exclusive lane and serves all left-turn movements. The middle lane group serves 239 veh/h of the 318 veh/h in the through movement (i.e., about 75%). The right lane group serves the remaining through vehicles (i.e., 79 veh/h) and the right-turning vehicles (106 veh/h) for a total flow rate of 185 veh/h. There are fewer vehicles in the right lane group (i.e., 185 versus 239) because some through drivers choose the middle lane to avoid any possible delay that might be incurred by the presence of right-turning vehicles in the outside lane.

Exhibit 18-46 summarizes the delay for each lane group, approach, and the intersection as a whole. It also provides the volume-to-capacity ratio and LOS for each lane group. The delay varies widely among lane groups, as does the LOS. The northbound through and right-turn movements have the highest delay and a LOS F condition.

Exhibit 18-46
Example Problem 1: Performance Measure Summary

Group:	EB Left L	EB Middle T	EB Right T+R	WB Left L	WB Middle T	WB Right T+R	NB Left L (Pr/Pm)	NB Middle T	NB Right T+R	SB Left L (Pr/Pm)	SB Middle T	SB Right T+R
Lane Group Summary												
Group Volume (v), veh/h	71.0	239.2	184.8	118.0	336.6	287.4	133.0	870.1	862.9	194.0	513.4	497.6
Volume-to-Capacity Ratio (X)	0.482	0.499	0.526	0.573	0.702	0.704	0.407	1.057	1.071	0.862	0.581	0.581
Control Delay (d), s/veh	45.846	30.017	30.729	43.979	35.832	36.617	13.547	73.592	78.392	34.020	17.094	17.116
Level of Service	D	C	C	D	D	D	B	F	F	C	B	B
Approach Summary												
Approach Volume, veh/h		495.0			742.0			1866.0			1205.0	
Approach Delay, s/veh		32.553			37.432			71.532			19.828	
Level of Service		C			D			E			B	
Intersection Summary												
Entering Volume, veh/h	4308.0											
Control Delay, s/veh	46.717											
Level of Service	D											

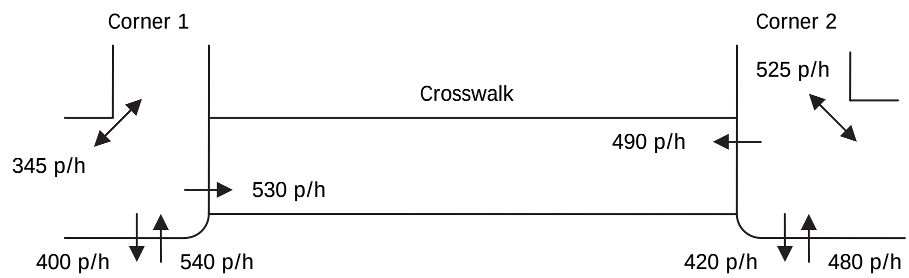
The fact that several phases are terminating by max-out and that the northbound through and right-turn movements are congested (i.e., Timer 8) suggests that some improvements could be made at this intersection. Simply increasing the maximum green settings is not a solution and, in fact, increases the overall delay and queue size for most lane groups. Physical changes to the intersection geometry to increase capacity could be considered.

EXAMPLE PROBLEM 2: PEDESTRIAN LOS

The Intersection

The pedestrian crossing of interest crosses the north leg at a signalized intersection. The north–south street is the minor street and the east–west street is the major street. The intersection serves all north–south traffic concurrently (i.e., no left-turn phases) and all east–west traffic concurrently. The signal has an 80-s cycle length. The crosswalk and intersection corners that are the subject of this example problem are shown in Exhibit 18-47.

Exhibit 18-47
Example Problem 2:
Pedestrian Flow Rates



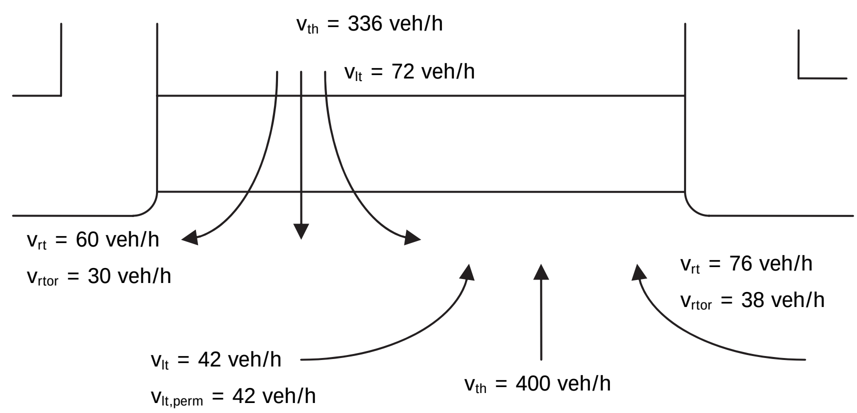
The Question

What is the pedestrian LOS for the crossing?

The Facts

Pedestrian flow rates are shown in Exhibit 18-47. Vehicular flow rates are shown in Exhibit 18-48.

Exhibit 18-48
Example Problem 2:
Vehicular Demand Flow
Rates



In addition, the following facts are known about the crosswalk and the intersection corners:

Major street: Phase duration, $D_{p,mj} = 48$ s
Yellow change interval, $Y_{mj} = 4$ s
Red clearance interval, $R_{mj} = 1$ s
Walk setting, $Walk_{mj} = 7$ s
Pedestrian clear setting, $PC_{mj} = 8$ s
Four traffic lanes (no turn bays)

Minor street: Phase duration, $D_{p,mi} = 32$ s
Yellow change interval, $Y_{mi} = 4$ s
Red clearance interval, $R_{mi} = 1$ s
Walk setting, $Walk_{mi} = 7$ s
Pedestrian clear setting, $PC_{mi} = 13$ s
Two traffic lanes (no turn bays)
85th percentile speed at a midsegment location, $S_{85,mi} = 35$ mi/h

Corner 1:	Total walkway width, $W_a = W_b = 16$ ft
	Corner radius, $R = 15$ ft
Corner 2:	Total walkway width, $W_a = W_b = 18$ ft
	Corner radius, $R = 15$ ft
Other data:	No right-turn channelizing islands provided on any corner
	Effective crosswalk width, $W_c = 16$ ft
	Crosswalk length, $L_c = 28$ ft
	Walking speed, $S_p = 4$ ft/s
	Pedestrian signal indications are provided for each crosswalk
	Rest-in-walk mode is not used for any phase

Comments

On the basis of the variable notation in Exhibit 18-25, the subject crosswalk is "Crosswalk C" because it crosses the minor street. The outbound pedestrian flow rate v_{co} at Corner 1 equals inbound flow rate v_{ci} at Corner 2, and the inbound flow rate v_{ci} at Corner 1 equals the outbound flow rate v_{co} at Corner 2.

Outline of Solution

First, the circulation area is calculated for both corners. Next, the circulation area is calculated for the crosswalk. The street corner and crosswalk circulation areas are then compared with the qualitative descriptions of pedestrian space listed in Exhibit 18-24.

Pedestrian delay and the pedestrian LOS score are then calculated for the crossing. Finally, LOS for the crossing is determined on the basis of the computed score and the threshold values in Exhibit 18-5.

Computational Steps

Step 1: Determine Street Corner Circulation Area

A. Compute Available Time–Space

For Corner 1, the available time–space is computed with Equation 18-52.

$$TS_{\text{corner}} = C(W_a W_b - 0.215R^2)$$

$$TS_{\text{corner}} = (80)[(16)(16) - 0.215(15)^2]$$

$$TS_{\text{corner}} = 16{,}610 \text{ ft}^2\text{-s}$$

B. Compute Holding-Area Waiting Time

Because pedestrian signal indications are provided and rest-in-walk is not enabled, the effective walk time for the phase serving the major street is computed with Equation 18-49.

$$g_{\text{Walk},mj} = Walk_{mj} + 4.0$$

$$g_{\text{Walk},mj} = 7.0 + 4.0 = 11 \text{ s}$$

The number of pedestrians arriving at the corner during each cycle to cross the minor street is computed with Equation 18-54.

$$N_{co} = \frac{v_{co}}{3{,}600} C$$

$$N_{co} = \frac{530}{3{,}600}(80) = 11.8 \text{ p}$$

The total time spent by pedestrians waiting to cross the minor street during one cycle is then calculated with Equation 18-53.

$$Q_{tco} = \frac{N_{co}(C - g_{\text{Walk},mj})^2}{2C}$$

$$Q_{tco} = \frac{(11.8)(80 - 11)^2}{2(80)} = 350.5 \text{ p-s}$$

By the same procedure, the total time spent by pedestrians waiting to cross the major street during one cycle (Q_{tdo}) is found to be 264.5 p-s.

C. Compute Circulation Time–Space

The circulation time–space is found by using Equation 18-58.

$$TS_c = TS_{\text{corner}} - [5.0(Q_{tdo} + Q_{tco})]$$

$$TS_c = 16{,}610 - [5.0\ (350.5 + 264.5)] = 13{,}535 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Corner Circulation Area

The total number of circulating pedestrians is computed with Equation 18-60.

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600} C$$

$$N_{tot} = \frac{490 + 530 + 540 + 400 + 345}{3{,}600}(80) = 51.2 \text{ p}$$

Finally, the corner circulation area per pedestrian is calculated with Equation 18-59.

$$M_{\text{corner}} = \frac{TS_c}{4.0 N_{tot}}$$

$$M_{\text{corner}} = \frac{13{,}535}{4.0(51.2)} = 66.1 \text{ ft}^2/\text{p}$$

By following the same procedure, the corner circulation area per pedestrian for Corner 2 is found to be 87.6 ft²/p. According to the qualitative descriptions provided in Exhibit 18-24, pedestrians at both corners will have the ability to move in the desired path, with no need to alter their movements to avoid conflicts.

Step 2: Determine Crosswalk Circulation Area

The analysis conducted in this step describes the circulation area for pedestrians in the subject crosswalk.

A. Establish Walking Speed

As given in the "facts" section, the average walking speed is determined to be 4.0 ft/s.

B. Compute Available Time–Space

Rest-in-walk is not enabled, so the pedestrian service time g_{ped} is estimated to equal the sum of the walk and pedestrian clear settings. The time–space available in the crosswalk is found with Equation 18-61.

$$TS_{cw} = L_c W_c \, g_{\text{Walk},mj}$$

$$TS_{cw} = (28)(16)(11) = 4{,}928 \text{ ft}^2\text{-s}$$

C. Compute Effective Available Time–Space

The number of turning vehicles during the walk and pedestrian clear intervals is calculated with Equation 18-64.

$$N_{tv} = \frac{v_{lt,perm} + v_{rt} - v_{rtor}}{3{,}600} C$$

$$N_{tv} = \frac{42 + 76 - 38}{3{,}600}(80) = 1.8 \text{ veh}$$

The time–space occupied by turning vehicles can then be computed with Equation 18-63.

$$TS_{tv} = 40 N_{tv} W_c$$

$$TS_{tv} = 40(1.8)(16) = 1{,}138 \text{ ft}^2\text{-s}$$

The effective available crosswalk time–space TS_{cw}* is found by subtracting the total available crosswalk time–space TS_{cw} from the time–space occupied by turning vehicles.

$$TS_{cw}^* = TS_{cw} - TS_{tv}$$

$$TS_{cw}^* = 4{,}928 - 1{,}138 = 3{,}790 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Service Time

The number of pedestrians exiting the curb when the WALK indication is presented is as follows:

$$N_{ped,co} = N_{co} \frac{C - g_{\text{Walk},mj}}{C}$$

$$N_{ped,co} = (11.8)\frac{80 - 11}{80} = 10.2 \text{ p}$$

Because the crosswalk width is greater than 10 ft, the pedestrian service time is computed as follows:

$$t_{ps,co} = 3.2 + \frac{L_c}{S_p} + 2.7\frac{N_{ped,co}}{W_c}$$

$$t_{ps,co} = 3.2 + \frac{28}{4.0} + 2.7\left(\frac{10.2}{16}\right) = 11.9 \text{ s}$$

The other travel direction in the crosswalk is analyzed next. The number of pedestrians arriving at Corner 1 each cycle by crossing the minor street is as follows:

$$N_{ci} = \frac{v_{ci}}{3{,}600}C$$

$$N_{ci} = \frac{490}{3{,}600}(80) = 10.9 \text{ p}$$

The sequence of calculations is repeated for this second travel direction in the subject crosswalk to indicate that $N_{ped,ci}$ is equal to 9.4 p and $t_{ps,ci}$ is 11.8.

E. Compute Crosswalk Occupancy Time

The crosswalk occupancy time for the crosswalk is computed as follows:

$$T_{occ} = t_{ps,co}N_{co} + t_{ps,ci}N_{ci}$$

$$T_{occ} = 11.9\ (11.8) + 11.8\ (10.9) = 268.6 \text{ p-s}$$

F. Compute Pedestrian Crosswalk Circulation Area

Finally, the crosswalk circulation area per pedestrian for the crosswalk is computed as follows:

$$M_{cw} = \frac{TS^*_{cw}}{T_{occ}}$$

$$M_{cw} = \frac{3{,}790}{268.6} = 14.1 \text{ ft}^2/\text{p}$$

The crosswalk circulation area is found to be 14.1 ft^2/p. According to the qualitative descriptions provided in Exhibit 18-24, pedestrians will find that their walking speed is restricted, with very limited ability to pass slower pedestrians. Improvements to the crosswalk should be considered and may include a wider crosswalk or a longer walk interval.

Step 3: Determine Pedestrian Delay

The pedestrian delay is calculated as follows:

$$d_p = \frac{(C - g_{\text{Walk},mj})^2}{2\,C}$$

$$d_p = \frac{(80-11)^2}{2\,(80)} = 29.8 \text{ s/p}$$

Step 4: Determine Pedestrian LOS Score for Intersection

The number of vehicles traveling on the minor street during a 15-min period is computed as follows:

$$n_{15,mi} = \frac{0.25}{N_c}\sum v_i$$

$$n_{15,mi} = \frac{0.25}{2}(72+336+60+42+400+76) = 123.3 \text{ veh/ln}$$

The cross-section adjustment factor is calculated as follows:

$$F_w = 0.681(N_c)^{0.514}$$

$$F_w = 0.681(2)^{0.514} = 0.972$$

The motorized vehicle adjustment factor is computed as follows:

$$F_v = 0.00569\left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,c}(0.0027 n_{15,mi} - 0.1946)$$

$$F_v = 0.00569\left(\frac{30+42}{4}\right) - (0)\,[0.0027(123.3) - 0.1946] = 0.102$$

The motorized vehicle speed adjustment factor is then computed:

$$F_s = 0.00013\; n_{15,mi}\, S_{85,mi}$$

$$F_s = 0.00013(123.3)(35) = 0.561$$

The pedestrian delay adjustment factor is calculated as follows:

$$F_{\text{delay}} = 0.0401 \ln(d_{p,c})$$

$$F_{\text{delay}} = 0.0401 \ln(29.8) = 0.136$$

The pedestrian LOS score for the intersection $I_{p,int}$ is then computed as follows:

$$I_{p,int} = 0.5997 + F_w + F_v + F_S + F_{\text{delay}}$$

$$I_{p,int} = 0.5997 + 0.972 + 0.102 + 0.561 + 0.136 = 2.37$$

For this crosswalk, $I_{p,int}$ is found to be 2.37.

Step 5: Determine LOS

According to Exhibit 18-5, the crosswalk operates at LOS B.

Discussion

The crosswalk was found to operate at LOS B in Step 5. It was determined in Step 1 that the pedestrians at both corners have adequate space to allow freedom of movement. Crosswalk circulation area was found to be restricted in Step 2 and improvements are probably justified. Moreover, the pedestrian delay computed in Step 3 was found to be slightly less than 30 s/p. With this much delay, some pedestrians may not comply with the signal indication.

EXAMPLE PROBLEM 3: BICYCLE LOS

The Intersection

A 5-ft-wide bicycle lane is provided at a signalized intersection.

The Question

What is the LOS of this bicycle lane?

The Facts

Saturation flow rate for bicycles = 2,000 bicycles/h

Effective green time = 48 s

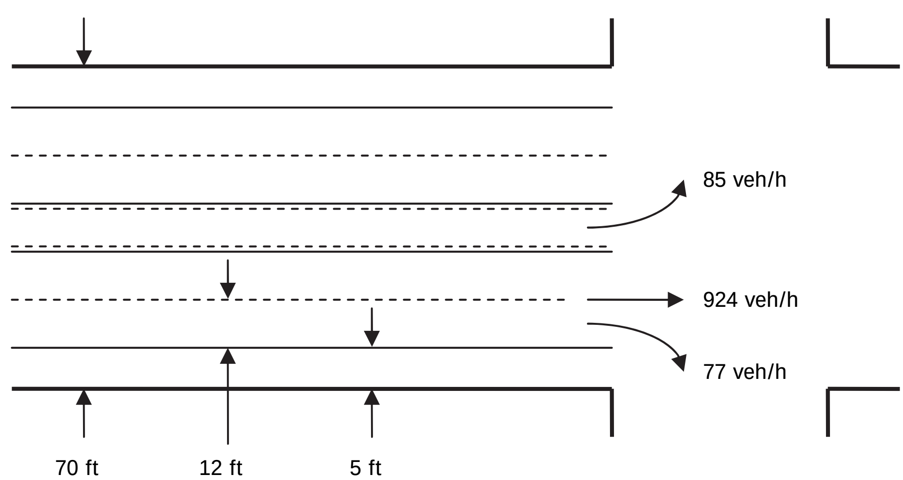
Cycle length = 120 s

Bicycle flow rate = 120 bicycles/h

No on-street parking

The vehicular flow rates and street cross-section element widths are as shown in Exhibit 18-49.

Exhibit 18-49
Example Problem 3: Vehicular Demand Flow Rates and Cross-Section Element Widths



Outline of Solution

Bicycle delay and the bicycle LOS score will be computed. LOS is then determined on the basis of the computed score and the threshold values in Exhibit 18-5.

Computational Steps

Step 1: Determine Bicycle Delay

A. Compute Bicycle Lane Capacity

The capacity of the bicycle lane is calculated with Equation 18-78:

$$c_b = s_b \frac{g_b}{C}$$

$$c_b = (2{,}000)\frac{48}{120} = 800 \text{ bicycles/h}$$

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 18-79:

$$d_b = \frac{0.5C(1 - g_b / C)^2}{1 - \dfrac{g_b}{C}\text{Min}\left[\dfrac{v_{bic}}{c_b}, 1.0\right]}$$

$$d_b = \frac{0.5(120)(1 - 48/120)^2}{1 - \dfrac{48}{120}\text{Min}\left[\dfrac{120}{800}, 1.0\right]} = 23.0 \text{ s/bicycle}$$

Step 2: Determine Bicycle LOS Score for Intersection

As shown in Exhibit 18-49, the total width of the outside through lane, bicycle lane, and paved shoulder is 17 ft (= 12 + 5 + 0). The cross-section adjustment factor can then be calculated with Equation 18-81:

$$F_w = 0.0153W_{cd} - 0.2144W_t$$

$$F_w = 0.0153(70) - 0.2144(17) = -2.57$$

The motor-vehicle volume adjustment factor must also be calculated, by using Equation 18-82:

$$F_v = 0.0066 \frac{v_{lt} + v_{th} + v_{rt}}{4N_{th}}$$

$$F_v = 0.0066 \frac{85 + 924 + 77}{4(2)} = 0.90$$

The bicycle LOS score can then be computed with Equation 18-80:

$$I_{b,int} = 4.1324 + F_w + F_v$$

$$I_{b,int} = 4.1324 - 2.57 + 0.90 = 2.45$$

Step 3: Determine LOS

According to Exhibit 18-5, this bicycle lane would operate at LOS B through the signalized intersection.

Discussion

The bicycle lane was found to operate at LOS B. The bicycle delay was found to be 23.0 s/bicycle, which is low enough that most bicyclists are not likely to be impatient. However, if the signal timing at the intersection were to be changed, the bicycle delay would need to be computed again to verify that it does not rise above 30 s/bicycle.

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Exhibit 19-16
TWSC Pedestrian Methodology

Step 1: Identify Two-Stage Crossings
↓
Step 2: Determine Critical Headway
↓
Step 3: Estimate Probability of a Delayed Crossing
↓
Step 4: Calculate Average Delay to Wait for Adequate Gap
↓
Step 5: Estimate Delay Reduction due to Yielding Vehicles
↓
Step 6: Calculate Average Pedestrian Delay and Determine LOS

Step 1: Identify Two-Stage Crossings

When a raised pedestrian-median refuge island is available, pedestrians typically cross in two stages, similar to the two-stage gap-acceptance described for automobiles earlier in this chapter. Determination of whether a pedestrian-median refuge exists may require engineering judgment. The main issue to determine is whether pedestrians cross the traffic streams in one or two stages. When pedestrians cross in two stages, pedestrian delay should be estimated separately for each stage of the crossing by using the procedures described in Steps 2 to 6. To determine pedestrian LOS, the pedestrian delay for each stage should be summed to establish the average pedestrian delay associated with the entire crossing. This service measure is used to determine pedestrian LOS for a TWSC intersection with two-stage crossings.

Step 2: Determine Critical Headway

Critical headway for pedestrians.

The procedure for estimating the critical headway is similar to that described for automobiles. The critical headway is the time in seconds below which a pedestrian will not attempt to begin crossing the street. Pedestrians use their judgment to determine whether the available headway between conflicting vehicles is long enough for a safe crossing. If the available headway is greater than the critical headway, it is assumed that the pedestrian will cross, but if the available headway is less than the critical headway, it is assumed that the pedestrian will not cross.

For a single pedestrian, critical headway is computed with Equation 19-69:

$$t_c = \frac{L}{S_p} + t_s$$

Equation 19-69

where

t_c = critical headway for a single pedestrian (s),

S_p = average pedestrian walking speed (ft/s),

L = crosswalk length (ft), and

t_s = pedestrian start-up time and end clearance time (s).

Pedestrian platooning.

If pedestrian platooning is observed in the field, then the spatial distribution of pedestrians should be computed with Equation 19-70. If no platooning is observed, the spatial distribution of pedestrians is assumed to be 1.

Equation 19-70

$$N_p = \text{Int}\left[\frac{8.0(N_c - 1)}{W_c}\right] + 1$$

where

N_p = spatial distribution of pedestrians (ped);

N_c = total number of pedestrians in the crossing platoon, from Equation 19-71 (ped);

W_c = crosswalk width (ft); and

8.0 = default clear effective width used by a single pedestrian to avoid interference when passing other pedestrians (ft).

To compute spatial distribution, the analyst must make field observations or estimate the platoon size by using Equation 19-71:

Equation 19-71

$$N_c = \frac{v_p e^{v_p t_c} + v e^{-v t_c}}{(v_p + v) e^{(v_p - v) t_c}}$$

where

N_c = total number of pedestrians in the crossing platoon (ped),

v_p = pedestrian flow rate (ped/s),

v = vehicular flow rate (veh/s), and

t_c = single pedestrian critical headway (s).

Group critical headway is determined with Equation 19-72:

Equation 19-72

$$t_{c,G} = t_c + 2(N_p - 1)$$

where

$t_{c,G}$ = group critical headway (s),

t_c = critical headway for a single pedestrian (s), and

N_p = spatial distribution of pedestrians (ped).

Step 3: Estimate Probability of a Delayed Crossing

On the basis of calculation of the critical headway t_G, the probability that a pedestrian will not incur any crossing delay is equal to the likelihood that a pedestrian will encounter a gap greater than or equal to the critical headway immediately upon arrival at the intersection.

Assuming random arrivals of vehicles on the major street, and equal distribution of vehicles among all through lanes on the major street, the probability of encountering a headway exceeding the critical headway in any given lane can be estimated by using a Poisson distribution. The likelihood that a gap in a given lane does not exceed the critical headway is thus the complement as shown in Equation 19-73. Because traffic is assumed to be distributed independently in each through lane, Equation 19-74 shows the probability that a pedestrian incurs nonzero delay at a TWSC crossing.

$$P_b = 1 - e^{\frac{-t_{c,G}v}{L}}$$

Equation 19-73

$$P_d = 1 - (1 - P_b)^L$$

Equation 19-74

where

P_b = probability of a blocked lane,

P_d = probability of a delayed crossing,

L = number of through lanes crossed,

$t_{c,G}$ = group critical headway (s), and

v = vehicular flow rate (veh/s).

Step 4: Calculate Average Delay to Wait for Adequate Gap

Research indicates that average delay to pedestrians at unsignalized crossings, assuming that no motor vehicles yield and the pedestrian is forced to wait for an adequate gap, depends on the critical headway, the vehicular flow rate of the subject crossing, and the mean vehicle headway (*10*). The average delay per pedestrian to wait for an adequate gap is given by Equation 19-75.

$$d_g = \frac{1}{v}\left(e^{vt_{c,G}} - vt_{c,G} - 1\right)$$

Equation 19-75

where

d_g = average pedestrian gap delay (s),

$t_{c,G}$ = group critical headway (s), and

v = vehicular flow rate (veh/s).

The average delay for any pedestrian who is unable to cross immediately upon reaching the intersection (e.g., any pedestrian experiencing nonzero delay) is thus a function of P_d and d_g, as shown in Equation 19-76:

$$d_{gd} = \frac{d_g}{P_d}$$

Equation 19-76

where

d_{gd} = average gap delay for pedestrians who incur nonzero delay,

d_g = average pedestrian gap delay (s), and

P_d = probability of a delayed crossing.

Step 5: Estimate Delay Reduction due to Yielding Vehicles

When a pedestrian arrives at a crossing and finds an inadequate gap, that pedestrian is delayed until one of two situations occurs: (*a*) a gap greater than the critical headway is available, or (*b*) motor vehicles yield and allow the pedestrian to cross. Equation 19-75 estimates pedestrian delay when motorists on the major approaches do not yield to pedestrians. Where motorist yield rates are significantly higher than zero, pedestrians will experience considerably less delay than that estimated by Equation 19-75.

In the United States, motorists are legally required to yield to pedestrians, under most circumstances, in both marked and unmarked crosswalks. However, actual motorist yielding behavior varies considerably. Motorist yield rates are influenced by a range of factors, including roadway geometry, travel speeds, pedestrian crossing treatments, local culture, and law enforcement practices.

Research (*11*, *12*) provides information on motorist responses to typical pedestrian crossing treatments, as shown in Exhibit 19-17. The exhibit shows results from two separate data collection methods. Staged data were collected with pedestrians trained by the research team to maintain consistent positioning, stance, and aggressiveness in crossing attempts. Unstaged data were collected through video recordings of the general population. The values shown in Exhibit 19-17 are based on a limited number of sites and do not encompass the full range of available crossing treatments. As always, practitioners should supplement these values with local knowledge and engineering judgment.

Exhibit 19-17
Effect of Pedestrian Crossing Treatments on Motorist Yield Rates

Crossing Treatment	Staged Pedestrians		Unstaged Pedestrians	
	Number of Sites	Mean Yield Rate, %	Number of Sites	Mean Yield Rate, %
Overhead flashing beacon (push button activation)	3	47	4	49
Overhead flashing beacon (passive activation)	3	31	3	67
Pedestrian crossing flags	6	65	4	74
In-street crossing signs (25–30 mi/h)	3	87	3	90
High-visibility signs and markings (35 mi/h)	2	17	2	20
High-visibility signs and markings (25 mi/h)	1	61	1	91
Rectangular rapid-flash beacon	N/A	N/A	17	81

Source: Fitzpatrick et al. (*11*) and Shurbutt et al. (*12*).

Depending on the crossing treatment and other factors, motorist behavior varies significantly.

It is possible for pedestrians to incur less actual delay than d_g because of yielding vehicles. The likelihood of this situation occurring is a function of vehicle volumes, motorist yield rates, and number of through lanes on the major street. Consider a pedestrian waiting for a crossing opportunity at a TWSC intersection, with vehicles in each conflicting through lane arriving every h seconds. On average, a potential yielding event will occur every h seconds, where $P(Y)$ represents the probability of motorists yielding for a given event. As

vehicles are assumed to arrive randomly, each potential yielding event is considered to be independent.

For any given yielding event, each through lane is in one of two states:

1. Clear—no vehicles are arriving within the critical headway window, or
2. Blocked—a vehicle is arriving within the critical headway window. The pedestrian may cross only if vehicles in each blocked lane choose to yield.

If not, the pedestrian must wait an additional h seconds for the next yielding event. On average, this process will be repeated until the wait exceeds the expected delay required for an adequate gap in traffic (d_{gd}), at which point the average pedestrian will receive an adequate gap in traffic and will be able to cross the street without having to depend on yielding motorists.

Thus, average pedestrian delay can be calculated with Equation 19-77, where the first term in the equation represents expected delay from crossings occurring when motorists yield, and the second term represents expected delay from crossings where pedestrians wait for an adequate gap.

$$d_p = \sum_{i=1}^{n} h(i-0.5)P(Y_i) + \left(P_d - \sum_{i=1}^{n} P(Y_i) \right) d_{gd}$$

Equation 19-77

where

d_p = average pedestrian delay (s),

i = crossing event ($i = 1$ to n),

h = average headway for each through lane,

$P(Y_i)$ = probability that motorists yield to pedestrian on crossing event i, and

n = Int(d_{gd}/h), average number of crossing events before an adequate gap is available.

Equation 19-77 requires the calculation of $P(Y_i)$. The probabilities $P(Y_i)$ that motorists will yield for a given crossing event are considered below for pedestrian crossings of one, two, three, and four through lanes.

One-Lane Crossing

Under the scenario in which a pedestrian crosses one through lane, $P(Y_i)$ is found simply. When $i = 1$, $P(Y_i)$ is equal to the probability of a delayed crossing P_d multiplied by the motorist yield rate, M_y. For $i = 2$, $P(Y_i)$ is equal to M_y multiplied by the probability that the second yielding event occurs (i.e., that the pedestrian did not cross on the first yielding event), $P_d^*(1 - M_y)$. Equation 19-78 gives $P(Y_i)$ for any i.

$$P(Y_i) = P_d M_y (1 - M_y)^{i-1}$$

Equation 19-78

where

M_y = motorist yield rate (decimal), and

i = crossing event ($i = 1$ to n).

Two-Lane Crossing

For a two-lane pedestrian crossing at a TWSC intersection, $P(Y_i)$ requires either (*a*) motorists in both lanes to yield simultaneously if both lanes are blocked, or (*b*) a single motorist to yield if only one lane is blocked. Because these cases are mutually exclusive, where $i = 1$, $P(Y_i)$ is equal to Equation 19-79:

Equation 19-79

$$P(Y_1) = 2P_b(1-P_b)M_y + P_b{}^2 M_y{}^2$$

Equation 19-80 shows $P(Y_i)$ where i is greater than 1. Equation 19-80 is equivalent to Equation 19-79 if $P(Y_0)$ is set to equal 0.

Equation 19-80

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{(2P_b(1-P_b)M_y) + (P_b^2 M_y^2)}{P_d}\right]$$

Three-Lane Crossing

A three-lane crossing follows the same principles as a two-lane crossing. Equation 19-81 shows the calculation for $P(Y_i)$:

Equation 19-81

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right] \times \left[\frac{P_b^3 M_y^3 + 3P_b^2(1-P_b)M_y^2 + 3P_b(1-P_b)^2 M_y}{P_d}\right]$$

where $P(Y_0) = 0$.

Four-Lane Crossing

A four-lane crossing follows the same principles as above. Equation 19-82 shows the calculation for $P(Y_i)$:

Equation 19-82

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right] \times$$
$$\left[\frac{P_b^4 M_y^4 + 4P_b^3(1-P_b)M_y^3 + 6P_b^2(1-P_b)^2 M_y^2 + 4P_b(1-P_b^3)M_y}{P_d}\right]$$

where $P(Y_0) = 0$.

Step 6: Calculate Average Pedestrian Delay and Determine LOS

The delay experienced by a pedestrian is the service measure. Exhibit 19-2 lists LOS criteria for pedestrians at TWSC intersections based on pedestrian delay. Pedestrian delay at TWSC intersections with two-stage crossings is equal to the sum of the delay for each stage of the crossing.

BICYCLE MODE

As of the publication date of this edition of the HCM, no methodology specific to bicyclists has been developed to assess the performance of bicyclists at TWSC intersections, as few data are available in the United States to support model calibration or LOS definitions. Depending on individual comfort level, ability, geometric conditions, and traffic conditions, bicyclists may travel through the intersection either as a motor vehicle or as a pedestrian. Critical headway

distributions have been identified in the research (*13*, *14*) for bicycles crossing two-lane major streets. Data on critical headways for bicycles under many circumstances are not readily available, however. Bicycles also differ from motor vehicles in that they normally do not queue linearly at a STOP sign. Instead, multiple bicycles often use the same gap in the vehicular traffic stream. This fact probably affects the determination of bicycle follow-up time. This phenomenon and others described in this section have not been adequately researched and are not explicitly included in the methodology.

3. APPLICATIONS

DEFAULT VALUES

A comprehensive presentation of potential default values for interrupted flow facilities is provided elsewhere (*15*), with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of *PHF* and percent heavy vehicles (%HV). Recommendations are based on geographic region, population, and time of day. All general default values for interrupted-flow facilities may be applied to the analysis of TWSC intersections in the absence of field data or projected conditions.

The following general default values may be applied to a TWSC intersection analysis:

- *PHF* = 0.92
- %HV = 3

Additional default values are sometimes required. For the analysis of shared or short major-street left-turn lanes, the following assumed default values may be applied for the saturation flow rates of the major-street through and right-turn movements:

- Major-street through movement, s_{i1} = 1,800 veh/h
- Major-street right-turn movement, s_{i2} = 1,500 veh/h

For analysis of pedestrians at TWSC intersections, the following default values may be applied:

- Average pedestrian walking speed, S_p = 3.5 ft/s
- Pedestrian start-up time and end clearance time, t_s = 3 s

As the number of default values used in any analysis increases, its accuracy becomes more approximate, and the result may be significantly different from the actual outcome, depending on local conditions.

ESTABLISH INTERSECTION BOUNDARIES

This methodology assumes that the TWSC intersection under investigation is isolated, with the exception of a TWSC intersection that is located within 0.25 mi of a signalized intersection (for the major-street approaches). When interaction effects are likely between the subject TWSC intersection and other intersections (e.g., queue spillback, demand starvation), the use of alternative tools may result in more accurate analysis. Analysis boundaries may also include different demand scenarios related to the time of day or to different development scenarios that produce various demand flow rates.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary engineering analysis.

Operational Analysis

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement in vehicles per hour, %HV for each approach, *PHF* for all demand volumes, lane configurations, specific geometric conditions, and upstream signal information. The outputs of an operational analysis are estimates of capacity, control delay, and queue lengths. The steps of the methodology, described in this chapter's methodology section, are followed directly without modification.

Design Analysis

The operational analysis described earlier in this chapter can be used for design purposes by using a given set of traffic flow data and iteratively determining the number and configuration of lanes that would be required to produce a given LOS.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of a TWSC intersection. To estimate LOS for a future time horizon, a planning analysis based on the operational method is used. The planning method uses all the geometric and traffic flow data required for an operational analysis, and the computations are identical. However, input variables for %HV and *PHF* are typically estimated (or defaults are used) when planning applications are performed.

Interpreting Results

Analysis of TWSC intersections is commonly performed to determine whether an existing intersection or driveway can remain as a TWSC intersection or whether additional treatments are necessary. These treatments, including geometric modifications and changes in traffic control, are discussed in other references, including the presentation of traffic signal warrants in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD; *16*). This section discusses two common situations analysts face: the analysis of shared versus separate lanes and the interpretation of LOS F.

Interpretation of the effects of shared lanes should take into account both delay associated with individual movements and delay associated with all vehicles on a given approach.

Some movements, most often left-turn movements, can sometimes have a poorer LOS when given a separate lane than when they share a lane with another movement (usually a through movement). This is not inconsistent in terms of the stated criteria. Left-turn movements will generally experience longer control delays than other movements because of the nature and priority of the movement. If left turns are placed in a shared lane, the control delay for vehicles in that lane may be less than the control delay for left turns in a separate lane. However, if delay for all vehicles on the approach or at the intersection is considered, providing separate lanes will result in lower total delay.

PERFORMANCE MEASURES

LOS F occurs when there are not enough gaps of suitable size to allow minor-street vehicles to enter or cross through traffic on the major street, resulting in long average control delays (greater than 50 s/veh). Depending on the demand on the approach, long queues on the minor approaches may result. The method, however, is based on a constant critical headway.

LOS F may also appear in the form of drivers on the minor street selecting smaller-than-usual gaps. In such cases, safety issues may occur, and some disruption to the major traffic stream may result. With lower demands, LOS F may not always result in long queues.

At TWSC intersections, the critical movement, often the minor-street left turn, may control the overall performance of the intersection. The lower threshold for LOS F is set at 50 s of delay per vehicle. In some cases, the delay equations will predict delays greater than 50 s for minor-street movements under very low-volume conditions on the minor street (fewer than 25 veh/h). On the basis of the first term of the delay equation, the LOS F threshold is reached with a movement capacity of approximately 85 veh/h or less, regardless of the minor-street movement volume.

This analysis procedure assumes random arrivals on the major street. For a typical major street with two lanes in each direction and an average traffic volume in the range of 15,000 to 20,000 veh/day (roughly equivalent to a peak hour flow rate of 1,500 to 2,000 veh/h), the delay equation will predict greater than 50 s of delay (LOS F) for many urban TWSC intersections that allow minor-street left-turn movements. LOS F will be predicted regardless of the volume of minor-street left-turning traffic. Even with an LOS F estimate, most low-volume minor-street approaches would not meet any of the MUTCD volume or delay warrants for signalization. As a result, analysts who use the HCM LOS thresholds to determine the design adequacy of TWSC intersections should do so with caution.

In evaluating the overall performance of TWSC intersections, it is important to consider measures of effectiveness in addition to delay, such as volume-to-capacity (v/c) ratios for individual movements, average queue lengths, and 95th percentile queue lengths. By focusing on a single measure of effectiveness for the worst movement only, such as delay for the minor-street left turn, users may make less effective traffic control decisions.

USE OF ALTERNATIVE TOOLS

Strengths of the HCM Procedure

This chapter offers a set of comprehensive procedures for analyzing the performance of an intersection under two-way STOP control. Simulation-based tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with the roadway and the control system, but for most purposes the HCM procedure produces an acceptable approximation.

The HCM procedure offers the advantage of a deterministic evaluation of a TWSC intersection, the results of which have been accepted by a broad

consensus of international experts. The HCM procedure also considers advanced concepts such as two-stage gap acceptance and flared approaches based on empirical evidence of their effects.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

The identified limitations for this chapter are shown in Exhibit 19-18, along with the potential for improved treatment by alternative tools.

Exhibit 19-18
Limitations of the HCM Signalized Intersection Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Effects of upstream intersections	Simulation tools can include an unsignalized intersection explicitly within a signalized arterial or network.
YIELD-controlled intersection operations	Treated explicitly by some tools. Can be approximated by varying the gap-acceptance parameters.
Non-steady-state conditions for demand and capacity	Most alternative tools provide for multiperiod variation of demand and, in some cases, capacity.
Macroscopic treatment of pedestrians and bicycles	Some simulation tools offer a microscopic modeling approach that provides explicit treatment of pedestrians and bicycles.

The most common application of alternative tools for TWSC involves an unsignalized intersection within a signalized arterial street.

Most analyses for isolated unsignalized intersections are intended to determine whether TWSC is a viable control alternative. Analyses of this type are handled adequately by the procedures described in this chapter. The main application for alternative tools at TWSC intersections involves coordinated arterial systems. Most intersections (i.e., those that are between the signals) operate under TWSC. These intersections tend to be ignored in the analysis of the system because their effect on the system operation is minimal. Occasionally, it is necessary to examine a TWSC intersection as a part of the arterial system. While the procedures in this chapter provide a method for approximating the operation of a TWSC intersection with an upstream signal, the operation of such an intersection is arguably best handled by including it in a complete simulation of the full arterial system. For example, queue backup from a downstream signal that blocks entry from the cross street for a portion of the cycle is not treated explicitly by the procedures contained in this chapter.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The performance measure that determines LOS for unsignalized intersections is *control delay,* defined as that portion of the delay that is due to the existence of the control device—in this case, a STOP sign. Most simulation tools do not produce explicit estimates of control delay.

The best way to determine control delay at a STOP sign from simulation is to perform simulation runs with and without the control device(s) in place. The segment delays reported with no control represent the delays due to geometrics and interaction between vehicles. The additional delay reported in the run with the control in place is, by definition, the control delay.

Chapter 7, Interpreting HCM and Alternative Tool Results, discusses performance measures from various tools in more detail, and Chapter 24, Concepts: Supplemental, provides recommendations on how individual vehicle trajectories should be interpreted to produce specific performance measures. Of

Delay and LOS should be estimated only by using alternative tools that conform to these definitions and computations of queue delay presented in this manual.

particular interest to TWSC operation is the definition of a "queued" state and the development of queue delay from that definition. For alternative tools that conform to the queue delay definitions and computations presented in this manual, the queue delay will provide the best estimate of control delay for TWSC intersections. Delay and LOS should not be estimated by using alternative tools that do not conform to these definitions and computations.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Deterministic tools and simulation tools both model TWSC operations as a gap-acceptance process that follows the rules of the road to determine the right-of-way hierarchy. To this extent, they are dealing in the same conceptual framework. Deterministic tools such as the HCM base their estimates of capacity and delay on expected values computed from analytical formulations that have been mathematically derived. Simulation tools take a more microscopic view, treating each vehicle as an independent object that is subject to the rules of the road as well as interaction with other vehicles. Differences in the treatment of randomness also exist, as explained in the Chapter 18, Signalized Intersections, guidance.

When the opposing movement volumes are very high, there is minimal opportunity for the STOP-controlled movements to accept gaps and these movements often have little or no capacity. Simulation tends to produce slightly higher capacities under these conditions because of overriding logic that limits the amount of time any driver is willing to wait for a gap. The overriding logic is somewhat tool specific.

In general, the simulation results for a specific TWSC intersection problem should be close to the results obtained from the procedures in this chapter. Some differences may, however, be expected among all the analysis tools.

Adjustment of Simulation Parameters to the HCM Parameters

The critical headways and follow-up headways are common to both deterministic and simulation models. It is therefore desirable that similar values be used for these parameters.

Sample Calculations Illustrating Alternative Tool Applications

It was mentioned previously that the most common application for TWSC simulation involves unsignalized intersections within a signalized arterial system. An example of this situation is presented in Chapter 29, Urban Street Facilities: Supplemental. An additional example involving blockage of a cross-street approach with STOP control by a queue from a nearby diamond interchange is presented in Chapter 34, Interchange Ramp Terminals: Supplemental.

4. EXAMPLE PROBLEMS

Exhibit 19-19
List of Example Problems

Example Problem	Title	Type of Analysis
1	TWSC T-intersection	Operational analysis
2	TWSC pedestrian crossing	Operational analysis

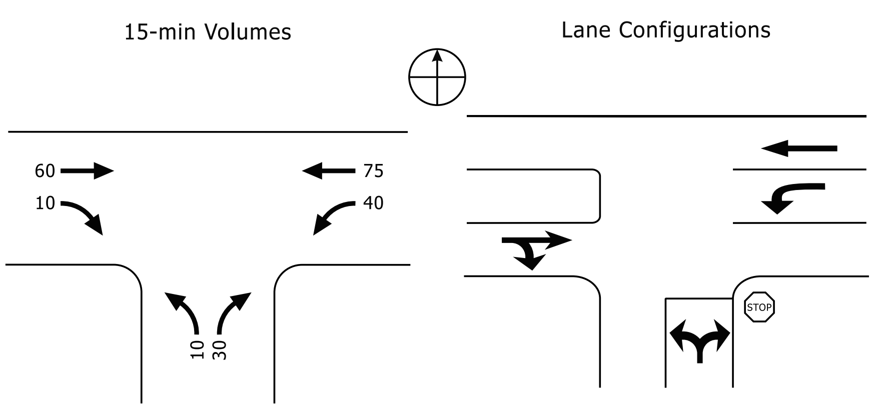
EXAMPLE PROBLEM 1: TWSC T-INTERSECTION

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- T-intersection,
- Major street with one lane in each direction,
- Minor street with one lane in each direction and STOP-controlled on the minor-street approach,
- Level grade on all approaches,
- Percent heavy vehicles on all approaches = 10%,
- No other unique geometric considerations or upstream signal considerations,
- No pedestrians,
- Length of analysis period = 0.25 h, and
- Volumes during the peak 15-min period and lane configurations as shown in Exhibit 19-20.

Exhibit 19-20
Example Problem 1 Movement Priorities, Lane Configurations, and Volumes



Comments

All input parameters are known, so no default values are needed or used.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Because peak 15-min volumes have been provided, each volume is multiplied by 4 to determine a peak 15-min flow rate (in veh/h) for each

movement. These values, along with the associated movement numbers, are shown in Exhibit 19-21.

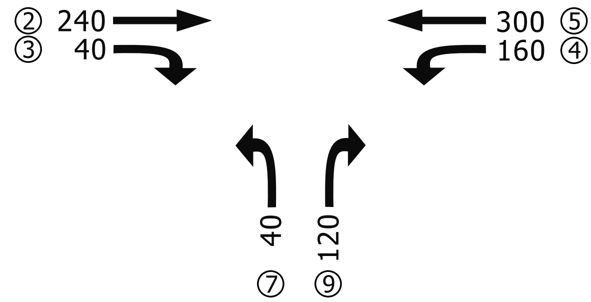


Exhibit 19-21
Example Problem 1: Calculation of Peak 15-min Flow Rates

Step 3: Compute Conflicting Flow Rates

The conflicting flow rates for each minor movement at the intersection are computed according to Equation 19-3, Equation 19-4, Equation 19-18, and Equation 19-24. The conflicting flow for the major-street left-turn $v_{c,4}$ is computed as follows:

$$v_{c,4} = v_2 + v_3 + v_{15}$$

$$v_{c,4} = 240 + 40 + 0 = 280 \text{ veh/h}$$

The conflicting flow for the minor-street right-turn movement $v_{c,9}$ is computed as follows:

$$v_{c,9} = v_2 + 0.5v_3 + v_{14} + v_{15}$$

$$v_{c,9} = 240 + 0.5(40) + 0 + 0 = 260 \text{ veh/h}$$

Finally, the conflicting flow for the minor-street left-turn movement $v_{c,7}$ is computed. Because two-stage gap acceptance is not present at this intersection, the conflicting flow rates shown in Stage I (Equation 19-18) and Stage II (Equation 19-24) are added together and considered as one conflicting flow rate. The conflicting flow for $v_{c,7}$ is computed as follows:

$$v_{c,7} = 2v_1 + v_2 + 0.5v_3 + v_{15} + 2v_4 + v_5 + 0.5v_6 + 0.5v_{12} + 0.5v_{11} + v_{13}$$

$$v_{c,7} = 2(0) + 240 + 0.5(40) + 0 + 2(160) + 300 + 0.5(0) + 0.5(0) + 0.5(0) + 0 = 880 \text{ veh/h}$$

Step 4: Determine Critical Headways and Follow-Up Headways

The critical headway for each minor movement is computed beginning with the base critical headway given in Exhibit 19-10. The base critical headway for each movement is then adjusted according to Equation 19-30. The critical headway for the major-street left-turn $t_{c,4}$ is computed as follows:

$$t_{c,4} = t_{c,base} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,4} = 4.1 + 1.0(0.1) + 0(0) - 0 = 4.2 \text{ s}$$

Similarly, the critical headway for the minor-street right-turn $t_{c,9}$ is computed as follows:

$$t_{c,9} = 6.2 + 1.0(0.1) + 0.1(0) - 0 = 6.3 \text{ s}$$

Finally, the critical headway for the minor-street left-turn $t_{c,7}$ is computed as follows:

$$t_{c,7} = 7.1 + 1.0(0.1) + 0.2(0) - 0.7 = 6.5 \text{ s}$$

The follow-up headway for each minor movement is computed beginning with the base follow-up headway given in Exhibit 19-11. The base follow-up headway for each movement is then adjusted according to Equation 19-31. The follow-up headway for the major-street left-turn $t_{f,4}$ is computed as follows:

$$t_{f,4} = t_{f,base} + t_{f,HV} P_{HV}$$

$$t_{f,4} = 2.2 + 0.9(0.1) = 2.29 \text{ s}$$

Similarly, the follow-up headway for the minor-street right-turn $t_{f,9}$ is computed as follows:

$$t_{f,9} = 3.3 + 0.9(0.1) = 3.39 \text{ s}$$

Finally, the follow-up headway for the minor-street left-turn $t_{f,7}$ is computed as follows:

$$t_{f,7} = 3.5 + 0.9(0.1) = 3.59 \text{ s}$$

Step 5: Compute Potential Capacities

The computation of a potential capacity for each movement provides the analyst with a definition of capacity under the assumed base conditions. The potential capacity will be adjusted in later steps to estimate the movement capacity for each movement. The potential capacity for each movement is a function of the conflicting flow rate, critical headway, and follow-up headway computed in the previous steps. The potential capacity for the major-street left-turn $c_{p,4}$ is computed as follows:

$$c_{p,4} = v_{c,4} \frac{e^{-v_{c,4} t_{c,4}/3{,}600}}{1 - e^{-v_{c,4} t_{f,4}/3{,}600}} = 280 \frac{e^{-(280)(4.2)/3{,}600}}{1 - e^{-(280)(2.29)/3{,}600}} = 1{,}238 \text{ veh/h}$$

Similarly, the potential capacity for the minor-street right-turn movement $c_{p,9}$ is computed as follows:

$$c_{p,9} = 260 \frac{e^{-(260)(6.3)/3{,}600}}{1 - e^{-(260)(3.39)/3{,}600}} = 760 \text{ veh/h}$$

Finally, the potential capacity for the minor-street left-turn movement $c_{p,7}$ is computed as follows:

$$c_{p,7} = 880 \frac{e^{-(880)(6.5)/3{,}600}}{1 - e^{-(880)(3.59)/3{,}600}} = 308 \text{ veh/h}$$

There are no upstream signals, so the adjustments for upstream signals are ignored.

Step 6: Compute Movement Capacities for Rank 1 Movements

There are no pedestrians at the intersection; therefore, all pedestrian impedance factors are equal to 1.0 and this step can be ignored.

Step 7: Compute Movement Capacities for Rank 2 Movements

The movement capacity for the major-street left-turn movement (Rank 2) $c_{m,4}$ is computed as follows:

$$c_{m,4} = (c_{p,4}) = 1{,}238 \text{ veh/h}$$

Similarly, the movement capacity for the minor-street right-turn movement (Rank 2) $c_{m,9}$ is computed as follows:

$$c_{m,9} = (c_{p,9}) = 760 \text{ veh/h}$$

Step 8: Compute Movement Capacities for Rank 3 Movements

The computation of vehicle impedance effects accounts for the reduction in potential capacity due to the impacts of the congestion of a high-priority movement on lower-priority movements.

Major-street movements of Rank 1 and Rank 2 are assumed to be unimpeded by other vehicular movements. Minor-street movements of Rank 3 can be impeded by major-street left-turn movements due to a major-street left-turning vehicle waiting for an acceptable gap at the same time as vehicles of Rank 3. The magnitude of this impedance depends on the probability that major-street left-turning vehicles will be waiting for an acceptable gap at the same time as vehicles of Rank 3. In this example, only the minor-street left-turn movement is defined as a Rank 3 movement. Therefore, the probability of the major-street left-turn operating in a queue-free state, $p_{0,4}$, is computed as follows:

$$p_{0,4} = 1 - \frac{v_4}{c_{m,4}} = 1 - \frac{160}{1{,}238} = 0.871$$

The movement capacity for the minor-street left-turn movement (Rank 3), $c_{m,7}$, is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor for the minor-street left-turn movement f_7 is computed with Equation 19-46 as follows:

$$f_7 = \prod_j p_{0,j} = 0.871$$

The movement capacity for the minor-street left-turn movement (Rank 3) $c_{m,7}$ is computed as follows:

$$c_{m,7} = (c_{p,7}) f_7 = (308)0.871 = 268 \text{ veh/h}$$

Step 9: Compute Movement Capacities for Rank 4 Movements

There are no Rank 4 movements in this example problem, so this step does not apply.

Step 10: Compute Capacity Adjustment Factors

In this example, the minor-street approach is a single lane shared by right-turn and left-turn movements; therefore, the capacity of these two movements must be adjusted to compute an approach capacity based on shared-lane effects.

The shared-lane capacity for the northbound minor-street approach $c_{SH,NB}$ is computed as follows:

$$c_{SH,NB} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)} = \frac{v_7 + v_9}{\frac{v_7}{c_{m,7}} + \frac{v_9}{c_{m,9}}} = \frac{40+120}{\frac{40}{268}+\frac{120}{760}} = 521 \text{ veh/h}$$

No other adjustments apply.

Step 11: Compute Control Delay

The control-delay computation for any movement includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The control delay for the major-street left-turn movement (Rank 2) d_4 is computed as follows:

$$d_4 = \frac{3600}{c_{m,4}} + 900T\left[\frac{v_4}{c_{m,4}} - 1 + \sqrt{\left(\frac{v_4}{c_{m,4}} - 1\right)^2 + \frac{\left(\frac{3,600}{c_{m,4}}\right)\left(\frac{v_4}{c_{m,4}}\right)}{450T}}\right] + 5$$

$$d_4 = \frac{3,600}{1,238} + 900(.25)\left[\frac{160}{1,238} - 1 + \sqrt{\left(\frac{160}{1,238} - 1\right)^2 + \frac{\left(\frac{3,600}{1,238}\right)\left(\frac{160}{1,238}\right)}{450(0.25)}}\right] + 5 = 8.3 \text{ s}$$

On the basis of Exhibit 19-1, the westbound left-turn movement is assigned LOS A.

The control delay for the minor-street right-turn and left-turn movements is computed by using the same formula; however, one significant difference from the major-street left-turn computation of control delay is that these movements share the same lane. Therefore, the control delay is computed for the approach as a whole and the shared-lane volume and shared-lane capacity must be used as follows:

$$d_{SH,NB} = \frac{3,600}{521} + 900(0.25)\left[\frac{160}{521} - 1 + \sqrt{\left(\frac{160}{521} - 1\right)^2 + \frac{\left(\frac{3,600}{521}\right)\left(\frac{160}{521}\right)}{450(0.25)}}\right] + 5 = 14.9 \text{ s}$$

On the basis of Exhibit 19-1, the northbound approach is assigned LOS B.

Step 11b: Compute Control Delay to Rank 1 Movements

This step is not applicable as the westbound major-street through movement v_5 and westbound major-street left-turn movement v_4 have exclusive lanes at this intersection. It is assumed that the eastbound through movement v_2 and eastbound major-street right-turn movement v_3 do not incur any delay at this intersection.

Step 11c: Compute Approach and Intersection Control Delay

The control delays to all vehicles on the eastbound approach are assumed to be negligible as described in Step 11b. The control delay for the westbound approach $d_{A,WB}$ is computed as follows:

$$d_{A,WB} = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

$$d_{A,WB} = \frac{0(0) + 0(300) + 8.3(160)}{0 + 300 + 160} = 2.9 \text{ s}$$

It is assumed that the westbound through movement incurs no control delay at this intersection. The control delay for the northbound approach was computed in Step 11a as $c_{SH,NB}$.

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB}}{v_{A,EB} + v_{A,WB} + v_{A,NB}}$$

$$d_I = \frac{0(280) + 2.9(460) + 14.9(160)}{280 + 460 + 160} = 4.1 \text{ s}$$

As noted previously, neither major-street approach LOS nor intersection LOS is defined.

Step 12: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for the major-street westbound left-turn movement, $Q_{95,4}$, is computed as follows:

$$Q_{95,4} \approx 900T \left[\frac{v_4}{c_{m,4}} - 1 + \sqrt{\left(\frac{v_4}{c_{m,4}} - 1 \right)^2 + \frac{\left(\frac{3600}{c_{m,4}} \right)\left(\frac{v_4}{c_{m,4}} \right)}{150T}} \right] \left(\frac{c_{m,4}}{3600} \right)$$

$$Q_{95,4} \approx 900(0.25) \left[\frac{160}{1238} - 1 + \sqrt{\left(\frac{160}{1238} - 1 \right)^2 + \frac{\left(\frac{3600}{1238} \right)\left(\frac{160}{1238} \right)}{150(0.25)}} \right] \left(\frac{1238}{3600} \right) = 0.4 \text{ veh}$$

The result of 0.4 veh for the 95th percentile queue indicates that a queue of more than one vehicle will occur very infrequently for the major-street left-turn movement.

The 95th percentile queue length for the northbound approach is computed by using the same formula. Similar to the control-delay computation, the shared-lane volume and shared-lane capacity must be used as shown:

$$Q_{95,NB} \approx 900(0.25)\left[\frac{160}{521} - 1 + \sqrt{\left(\frac{160}{521} - 1\right)^2 + \frac{\left(\frac{3{,}600}{521}\right)\left(\frac{160}{521}\right)}{150(0.25)}}\right]\left(\frac{521}{3{,}600}\right) = 1.3 \text{ veh}$$

The result suggests that a queue of more than one vehicle will occur only occasionally for the northbound approach.

Discussion

Overall, the results indicate that the three-leg, TWSC intersection will operate well with small delays and little queuing for all minor movements.

EXAMPLE PROBLEM 2: TWSC PEDESTRIAN CROSSING

Calculate the pedestrian LOS of a pedestrian crossing of a major street at a TWSC intersection under the following circumstances:

- Scenario A: Unmarked crosswalk, no median refuge island;
- Scenario B: Unmarked crosswalk, median refuge island; and
- Scenario C: Marked crosswalk with high-visibility treatments, median refuge island.

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Four-lane major street;
- 1,700 peak hour vehicles, bidirectional;
- Crosswalk length without median = 46 ft;
- Crosswalk length with median = 40 ft;
- Observed pedestrian walking speed = 4 ft/s;
- Pedestrian start-up time = 3 s; and
- No pedestrian platooning.

Comments

In addition to the input data listed above, information is required on motor vehicle yield rates under the various scenarios. On the basis of an engineering study of similar intersections in the vicinity, it is determined that motor vehicle yield rates are 0% with unmarked crosswalks and 50% with high-visibility marked crosswalks.

Step 1: Identify Two-Stage Crossings

Scenario A does not have two-stage pedestrian crossings, as no median refuge is available. Analysis for Scenarios B and C should assume two-stage crossings. Thus, analysis for Scenarios B and C will combine two equidistant pedestrian crossings of 20 ft to determine the total delay.

Step 2: Determine Critical Headway

Because there is no pedestrian platooning, the critical headway is determined with Equation 19-69:

Scenario A: t_c = (46 ft/4 ft/s) + 3 s = 14.5 s

Scenario B: t_c = (20 ft/4 ft/s) + 3 s = 8 s

Scenario C: t_c = (20 ft/4 ft/s) + 3 s = 8 s

Step 3: Estimate Probability of a Delayed Crossing

Equation 19-73 and Equation 19-74 are used to calculate P_b, the probability of a blocked lane, and P_d, the probability of a blocked crossing, respectively. In the case of Scenario A, the crossing consists of four lanes. Scenarios B and C have only two lanes, given the two-stage crossing opportunity.

$$P_b = 1 - e^{\frac{-t_{c,G} v}{L}}$$

$$P_d = 1 - (1 - P_b)^L$$

where

P_b = probability of a blocked lane,

P_d = probability of a delayed crossing,

L = number of through lanes crossed,

$t_{c,G}$ = group critical headway (s), and

v = vehicular flow rate (veh/s).

For the single-stage crossing, v is (1,700 veh/h)/(3,600 s/h) = 0.47 veh/s.

For the two-stage crossing, without any information on directional flows, one-half the volume is used, and v is therefore (850 veh/h)/(3,600 s/h) = 0.24 veh/s.

Scenario A:

$$P_b = 1 - e^{\frac{-14.5 \times 0.47}{4}} = 0.82$$

$$P_d = 1 - (0.18)^4 = 0.999$$

Scenario B:

$$P_b = 1 - e^{\frac{-8 \times 0.24}{2}} = 0.61$$

$$P_d = 1 - (0.39)^2 = 0.85$$

Scenario C:

$$P_b = 1 - e^{\frac{-8\times 0.24}{2}} = 0.61$$

$$P_d = 1 - (0.39)^2 = 0.85$$

Step 4: Calculate Average Delay to Wait for Adequate Gap

Average gap delay d_g and average gap delay when delay is nonzero d_{gd} are calculated by Equation 19-75 and Equation 19-76.

Scenario A:

$$d_g = \frac{1}{0.47} \times \left(e^{0.47\times 14.5} - 0.47\times 14.5 - 1\right) = 1{,}977 \text{ s}$$

$$d_{gd} = \frac{1{,}977}{0.999} = 1{,}979 \text{ s}$$

Scenario B:

$$d_g = \frac{1}{0.24}\left(e^{0.24\times 8} - 0.24\times 8 - 1\right) = 15.8 \text{ s}$$

$$d_{gd} = \frac{15.8}{0.85} = 18.6 \text{ s}$$

Scenario C:

$$d_g = \frac{1}{0.24}\left(e^{0.24\times 8} - 0.24\times 8 - 1\right) = 15.8 \text{ s}$$

$$d_{gd} = \frac{15.8}{0.85} = 18.6 \text{ s}$$

Step 5: Estimate Delay Reduction due to Yielding Vehicles

Under Scenarios A and B, the motorist yield rates are approximately 0%. Therefore, there is no reduction in delay due to yielding vehicles, and average delay is the same as that shown in Step 4. Under Scenario C, motorist yield rates are 50%. Because of the two-stage crossing, use Equation 19-80 to determine $P(Y_i)$:

$$P(Y_1) = [0.85 - 0]\left[\frac{(2\times 0.61(1-0.61)0.50) + (0.61^2 0.50^2)}{0.85}\right] = 0.33$$

$$P(Y_2) = [0.85 - 0.33]\left[\frac{(2\times 0.61(1-0.61)0.50) + (0.61^2 0.50^2)}{0.85}\right] = 0.20$$

The results of Equation 19-80 can be substituted into Equation 19-77 to determine average pedestrian delay.

$$d_p = \sum_{i=1}^{2} 8.5(i - 0.5)P(Y_i) + \left(0.85 - \sum_{i=1}^{2} P(Y_i)\right)18.6 = 9.8\ \mathrm{s}$$

Step 6: Calculate LOS

Average pedestrian delays and pedestrian LOS under each of the three scenarios are as follows:

Scenario A = 1,979 s = LOS F

Scenario B = 2 × 15.8 s = 31.6 s = LOS E

Scenario C = 2 × 9.8 s = 19.6 s = LOS C

5. REFERENCES

Some of these references are available in the Technical Reference Library in Volume 4.

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CHAPTER 20
ALL-WAY STOP-CONTROLLED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

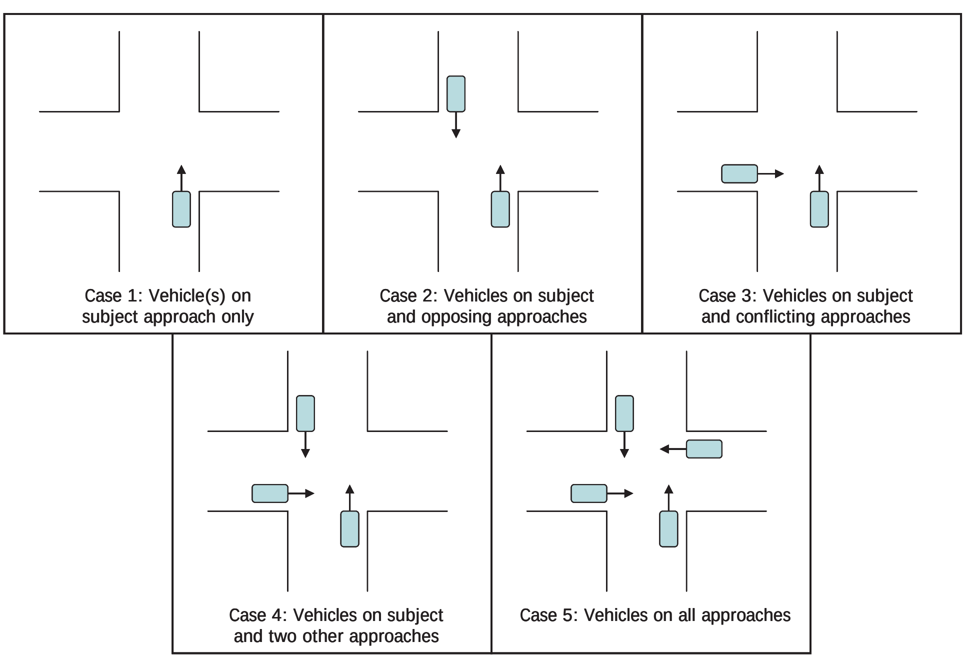
Chapter 20, All-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing these types of intersections (*1*). A glossary and list of symbols, including those used for all-way STOP-controlled (AWSC) intersections, is provided in Chapter 9.

AWSC intersections require every vehicle to stop at the intersection before proceeding. Because each driver must stop, the decision to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after stopping. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that no vehicles are currently in the intersection and that it is the driver's turn to proceed.

Field observations indicate that standard four-leg AWSC intersections operate in either a two-phase or a four-phase pattern, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north–south and east–west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach intersection).

If traffic is present on the subject approach only, vehicles depart as rapidly as individual drivers can safely accelerate into and clear the intersection. This case is illustrated as Case 1 in Exhibit 20-1.

Exhibit 20-1
Analysis Cases for AWSC Intersections



If traffic is present on the other approaches, as well as on the subject approach, the saturation headway (the time between subsequent vehicle departures) on the subject approach will increase somewhat, depending on the degree of conflict that results between the subject approach vehicles and the

vehicles on the other approaches. In Case 2, some uncertainty is introduced with a vehicle on the opposing approach, and thus the saturation headway will be greater than for Case 1. In Case 3, vehicles on one of the conflicting approaches further restrict the departure rate of vehicles on the subject approach, and the saturation headway will be longer than for Case 1 or Case 2. In Case 4, two vehicles are waiting on opposing or conflicting approaches, and saturation headways are even longer. When vehicles are present on all approaches, as in Case 5, saturation headways are the longest of any of the cases because the potential for conflict between vehicles is greatest. The increasing degree of potential conflict translates directly into longer driver decision times and longer saturation headways. Because no traffic signal controls the stream movement or allocates the right-of-way to each conflicting traffic stream, the rate of departure is controlled by the interactions between the traffic streams.

Capacity of an AWSC can be described by saturation headway, departure headway, and service time.

Therefore, the operation at an AWSC intersection can be described numerically by a few key time-based terms:

- The saturation headway, h_{si}, is the time between departures of successive vehicles on a given approach for a particular case (case i), as described above, assuming a continuous queue.
- The departure headway, h_d, is the average time between departures of successive vehicles on a given approach accounting for the probability of each possible case.
- The service time, t_s, is the average time spent by a vehicle in first position waiting to depart. It is equal to the departure headway minus the time it takes a vehicle to move from second position into first position (the move-up time, m).

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The intersection analysis boundaries for an AWSC analysis are assumed to be those of an isolated intersection; that is, no upstream or downstream effects are accounted for in the analysis. The present methodology is limited to motor vehicles.

LEVEL-OF-SERVICE CRITERIA

The level-of-service (LOS) criteria for AWSC intersections are given in Exhibit 20-2. As the exhibit notes, LOS F is assigned if the volume-to-capacity (v/c) ratio of a lane exceeds 1.0, regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

Exhibit 20-2
LOS Criteria: Automobile Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio*	
	$v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: * For approaches and intersectionwide assessment, LOS is defined solely by control delay.

REQUIRED INPUT DATA

Analysis of an AWSC intersection requires the following data:

1. Number and configuration of lanes on each approach;
2. Percentage of heavy vehicles;
3. Turning movement demand flow rate for each entering lane or, alternatively, hourly demand volume and peak hour factor; and
4. Length of analysis period—generally a peak 15-min period within the peak hour, although any 15-min period can be analyzed.

SCOPE OF THE METHODOLOGY

This chapter focuses on the operation of AWSC intersections. This version of the AWSC intersection analysis procedures is primarily a result of studies conducted by National Cooperative Highway Research Program Project 3-46 (*1*).

LIMITATIONS OF THE METHODOLOGY

Automobile Mode

The methodologies in this chapter apply to isolated AWSC intersections with up to three lanes on each approach. They do not account for interaction effects with other intersections. The methodologies do not apply to AWSC intersections with more than four approaches. In addition, the effect of conflicting pedestrians on automobiles is not considered in this procedure. Conflicting pedestrian movements are likely to increase the saturation headway of affected vehicular movements, but the magnitude of this effect is unknown as of the publication of this edition of the HCM.

Pedestrian and Bicycle Modes

The current methodologies for analyzing LOS and delay at AWSC intersections do not extend to pedestrians and apply to bicycles only in limited situations that are not supported by research at the time of publication of this edition. As such, there are no set LOS standards that apply to pedestrians or bicycles at AWSC intersections, nor can pedestrian or bicycle delay, capacity, or quality of service be quantitatively assessed by using the procedures described in this chapter. Additional research on pedestrian and bicyclist behavior and operations at AWSC intersections needs to be done before procedures can be developed that adequately address these issues. A discussion of qualitative effects is included in the methodology section of this chapter.

2. METHODOLOGY

OVERVIEW

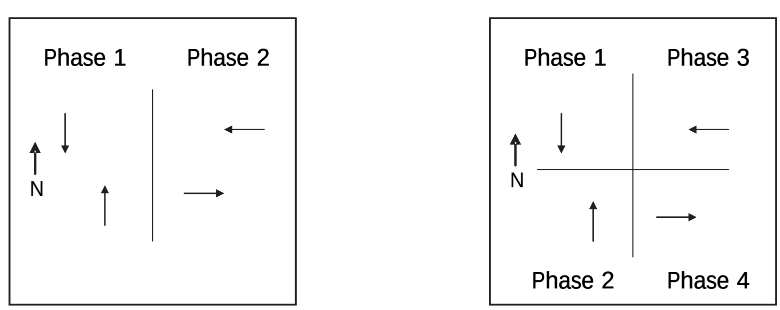
AWSC intersections require drivers on all approaches to stop before proceeding into the intersection. While giving priority to the driver on the right is a recognized rule in some areas, it is not a good descriptor of actual intersection operations. What happens is the development of a consensus of right-of-way that alternates between the drivers on the intersection approaches, a consensus that depends primarily on the intersection geometry and the arrival patterns at the stop line.

The methodology analyzes each intersection approach independently. The approach under study is called the subject approach. The opposing approach and the conflicting approaches create conflicts with vehicles on the subject approach.

Phase Patterns

A two-phase pattern, as shown in Exhibit 20-3(a), is observed at a standard four-leg AWSC intersection (one approach lane on each leg), where drivers from opposing approaches enter the intersection at roughly the same time. Some interruption of this pattern occurs when there are conflicts between certain turning maneuvers (such as a northbound left-turning vehicle and a southbound through vehicle), but generally the north–south streams alternate right-of-way with the east–west streams. A four-phase pattern, as shown in Exhibit 20-3(b), emerges at multilane four-leg intersections, where development of the right-of-way consensus is more difficult. Here drivers from each approach enter the intersection together as right-of-way passes from one approach to the next and each is served in turn. A similar three-phase pattern emerges at multilane three-leg intersections.

Exhibit 20-3
Operation Patterns at AWSC Intersections



(a) Two-phase (single-lane approaches) (b) Four-phase (multilane approaches)

Two cases for departure headways.

The headways of vehicles departing from the subject approach fall into one of two cases. If there are no vehicles on any of the other approaches, subject approach vehicles can enter the intersection immediately after stopping. However, if vehicles are waiting on a conflicting approach, a vehicle from the subject approach must wait for consensus with the next conflicting vehicle. The headways between consecutively departing subject approach vehicles will be shorter in the first case than in the second case. Thus, the headway for a departing subject approach vehicle depends on the degree of conflict experienced

with vehicles on the other intersection approaches. The degree of conflict increases with two factors: the number of vehicles on the other approaches and the complexity of the intersection geometry.

Vehicle type and turning movement affect departure headway. These effects are captured empirically in the method.

Two other factors affect the departure headway of a subject approach vehicle: vehicle type and turning movement. The headway for a heavy vehicle will be longer than that for a passenger car. Furthermore, the headway for a left-turning vehicle will be longer than that for a through vehicle, which in turn will be longer than that for a right-turning vehicle.

In summary:

1. Standard four-leg AWSC intersections operate in either two-phase or four-phase patterns, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north–south and east–west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach).
2. The headways between consecutively departing subject approach vehicles depend on the degree of conflict between these vehicles and the vehicles on the other intersection approaches. The degree of conflict is a function of the number of vehicles faced by the subject approach vehicle and of the number of lanes on the intersection approaches.
3. The headway of a subject approach vehicle also depends on its vehicle type and its turning maneuver (if any).

Capacity Concepts

Capacity defined.

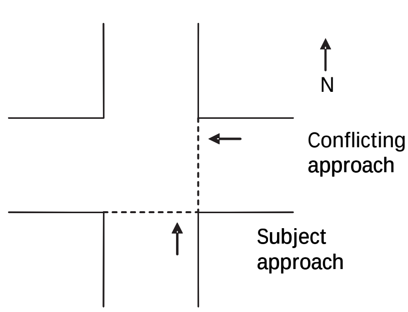
Capacity is defined as the maximum throughput on an approach given the flow rates on the other intersection approaches. The capacity model described here is an expansion of earlier work (2). The model is described for four increasingly complex cases: the intersection of two one-way streets with no turning movements, the intersection of two two-way streets with no turning movements, a generalized model for single-lane sites, and a generalized model for multilane sites. The methodology described later in this chapter is an implementation of the latter and most general case.

Intersection of Two One-Way Streets

The impact of turning movements is considered later, as part of the generalized models.

The first formulation of the model is based on the intersection of two one-way streets, each STOP-controlled. In this basic model, vehicles on either approach travel only straight through the intersection, as shown in Exhibit 20-4.

Exhibit 20-4
AWSC Configuration: Formulation 1



The saturation headway for a vehicle assumes one of two values: h_{s1} is the saturation headway if no vehicle is waiting on the conflicting approach, and h_{s2} is the saturation headway if the conflicting approach is occupied. The departure headway for vehicles on an approach is the expected value of this bivalued distribution. For the northbound approach, the mean service time is computed by Equation 20-1:

Equation 20-1

$$h_{d,N} = h_{s1}(1 - x_W) + h_{s2}x_W$$

where x_W is the degree of utilization of the westbound approach and is equal to the probability of finding at least one vehicle on that approach. Thus $1 - x_W$ is the probability of finding no vehicle on the westbound approach.

By symmetry, the mean service time for the westbound approach is given by Equation 20-2.

Equation 20-2

$$h_{d,W} = h_{s1}(1 - x_N) + h_{s2}x_N$$

Since the degree of utilization x is the product of the arrival rate λ and the mean departure headway h_d, the departure headways for each approach can be expressed in terms of the bivalued saturation headways and the arrival rates on each approach, as in Equation 20-3 and Equation 20-4.

Equation 20-3

$$h_{d,N} = \frac{h_{s1}\left[1 + \lambda_W\left(h_{s2} - h_{s1}\right)\right]}{1 - \lambda_N\lambda_W\left(h_{s2} - h_{s1}\right)^2}$$

Equation 20-4

$$h_{d,W} = \frac{h_{s1}\left[1 + \lambda_N\left(h_{s2} - h_{s1}\right)\right]}{1 - \lambda_N\lambda_W\left(h_{s2} - h_{s1}\right)^2}$$

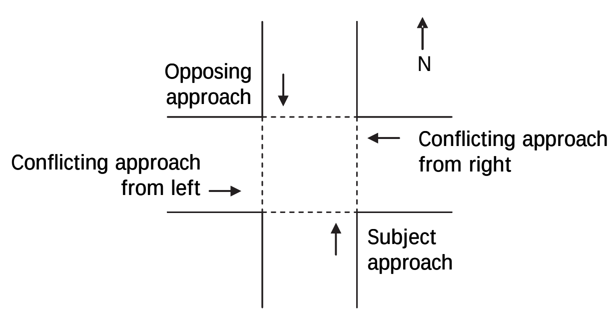
Intersection of Two Two-Way Streets

In this simplified model, the saturation headway for a vehicle assumes one of two values, h_{s1} or h_{s2}, because vehicles are again assumed to pass straight through the intersection. The departure headway for vehicles on an approach is the expected value of this bivalued distribution. A northbound vehicle will have a saturation headway of h_{s1} if the eastbound and westbound approaches are empty simultaneously. The probability of this event is the product of the probability of an empty westbound approach and the probability of an empty eastbound approach. The departure headway for the northbound vehicle is computed with Equation 20-5. See Exhibit 20-5.

Equation 20-5

$$h_{d,N} = h_{s1}\left(1 - x_E\right)\left(1 - x_W\right) + h_{s2}\left[1 - \left(1 - x_E\right)\left(1 - x_W\right)\right]$$

Exhibit 20-5
AWSC Configuration: Formulation 2



Unlike Formulation 1, it is not possible to solve directly for the departure headway in terms of a combination of arrival rates and the bivalued saturation headways. The departure headway on any approach depends on, or is directly coupled with, the traffic intensity on the two conflicting approaches. This coupling prevents a direct solution. However, it is possible to solve for the departure headway on each approach in an iterative manner, by using a system of equations similar in form to Equation 20-5.

Generalized Model for Single-Lane Sites

Capacity is determined by an iterative procedure.

The generalized model is based on five saturation headway values, each reflecting a different level or degree of conflict faced by the subject approach driver. Exhibit 20-6 specifies the conditions for each case and the probability of occurrence of each. The probability of occurrence is based on the degree of utilization on the opposing and conflicting approaches. The essence of the model, and its complexity, is evident when one realizes that the traffic intensity on one approach is computed from its capacity, which in turn depends on the traffic intensity on the other approaches. The interdependence of the traffic flow on all intersection approaches creates the need for iterative calculations to obtain stable estimates of departure headway and service time—and thus capacity.

Exhibit 20-6
Probability of Degree-of-Conflict Case

Degree-of-Conflict Case	Approach Opp	Con-L	Con-R	Probability of Occurrence
1	N	N	N	$(1-x_O)(1-x_{CL})(1-x_{CR})$
2	Y	N	N	$(x_O)(1-x_{CL})(1-x_{CR})$
3	N	Y	N	$(1-x_O)(x_{CL})(1-x_{CR})$
3	N	N	Y	$(1-x_O)(1-x_{CL})(x_{CR})$
4	Y	N	Y	$(x_O)(1-x_{CL})(x_{CR})$
4	Y	Y	N	$(x_O)(x_{CL})(1-x_{CR})$
4	N	Y	Y	$(1-x_O)(x_{CL})(x_{CR})$
5	Y	Y	Y	$(x_O)(x_{CL})(x_{CR})$

Note: Opp = opposing approach, Con-L = conflicting approach from the left, Con-R = conflicting approach from the right, N = no, Y = yes.

The probability, $P(C_i)$, for each degree-of-conflict case given in Exhibit 20-6 can be computed with Equation 20-6 through Equation 20-10. The degrees of utilization on the opposing approach, the conflicting approach from the left, and the conflicting approach from the right are given by x_O, x_{CL}, and x_{CR}, respectively.

Equation 20-6
$$P(C_1)=(1-x_O)(1-x_{CL})(1-x_{CR})$$

Equation 20-7
$$P(C_2)=(x_O)(1-x_{CL})(1-x_{CR})$$

Equation 20-8
$$P(C_3)=(1-x_O)(x_{CL})(1-x_{CR})+(1-x_O)(1-x_{CL})(x_{CR})$$

Equation 20-9
$$P(C_4)=(x_O)(1-x_{CL})(x_{CR})+(x_O)(x_{CL})(1-x_{CR})+(1-x_O)(x_{CL})(x_{CR})$$

Equation 20-10
$$P(C_5)=(x_O)(x_{CL})(x_{CR})$$

The departure headway for an approach is the expected value of the saturation headway distribution, computed by Equation 20-11:

Equation 20-11

$$h_d = \sum_{i=1}^{5} P(C_i) h_{si}$$

where $P(C_i)$ is the probability of the degree-of-conflict case C_i and h_{si} is the saturation headway for that case, given the traffic stream and geometric conditions of the intersection approach.

Capacity is determined by increasing volume on the subject approach until x > 1.0.

The capacity is computed by incrementally increasing the volume on the subject approach until the degree of utilization exceeds 1.0. This flow rate is the maximum possible flow or throughput on the subject approach under the conditions used as input to the analysis.

Generalized Model for Multilane Sites

Saturation headways at multilane sites are typically longer than at single-lane sites, all other conditions being equal. This situation is the result of two factors:

- A larger intersection (i.e., greater number of lanes) requires more travel time through the intersection, thus increasing the saturation headway; and
- Additional lanes also result in an increasing degree of conflict with opposing and conflicting vehicles, again increasing driver decision time and the saturation headway.

By contrast, some movements may not conflict with each other as readily at multilane sites as at single-lane sites. For example, a northbound vehicle turning right may be able to depart simultaneously with an eastbound through movement if the two vehicles are able to occupy separate receiving lanes when departing to the east. Consequently, in some cases, the saturation headway may be lower at multilane sites.

The theory described earlier proposed that the saturation headway is a function of the directional movement of the vehicle, the vehicle type, and the degree of conflict faced by the subject vehicle. This theory is extended here for multilane sites with respect to the concept of degree of conflict: saturation headway is affected to a large extent by the number of opposing and conflicting vehicles faced by the subject driver. For example, in degree-of-conflict Case 2, a subject vehicle is faced only by a vehicle on the opposing approach. At a two-lane approach intersection, there can be either one or two vehicles on the opposing approach. Each degree-of-conflict case is expanded to consider the number of vehicles present on each of the opposing and conflicting approaches. The cases are defined in Exhibit 20-7 and Exhibit 20-8 for two-lane and three-lane approaches, respectively.

For multilane sites, separate saturation headway values are computed for the number of vehicles faced by the subject vehicle for each degree-of-conflict case. This calculation requires a further extension of the service time model to account for the increased number of subcases. These combinations can be further subdivided if a vehicle can be present on any lane on a given approach.

Exhibit 20-7
Degree-of-Conflict Cases for Two-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2
3		x		1, 2
			x	
4	x	x		2, 3, 4
	x		x	
		x	x	
5	x	x	x	3, 4, 5, 6

Exhibit 20-8
Degree-of-Conflict Cases for Three-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2, 3
3		x		1, 2, 3
			x	
4	x	x		2, 3, 4, 5, 6
	x		x	
		x	x	
5	x	x	x	3, 4, 5, 6, 7, 8, 9

The probability of a vehicle being at the stop line in a given lane is x, the degree of utilization. The product of the six degrees of saturation, encompassing each of the six lanes on the opposing or conflicting approaches (two lanes on the opposing approach and two lanes on each of the conflicting approaches), gives the probability of any particular combination occurring.

The iterative procedure to compute the departure headways and capacities for each approach as a function of the departure headways on the other approaches is the same as described earlier. However, the additional subcases clearly increase the complexity of this computation.

AUTOMOBILE MODE

The AWSC intersection methodology for the automobile mode is applied through a series of steps that relate to input data, saturation headways, departure headways, service time, capacity, and LOS. They are illustrated in Exhibit 20-9.

Step 1: Convert Movement Demand Volumes to Flow Rates

Flow rates for each turning movement at the intersection must be converted from hourly volumes in vehicles per hour (veh/h) to peak 15-min flow rates in vehicles per hour as given in Equation 20-12:

Equation 20-12

$$v_i = \frac{V_i}{PHF}$$

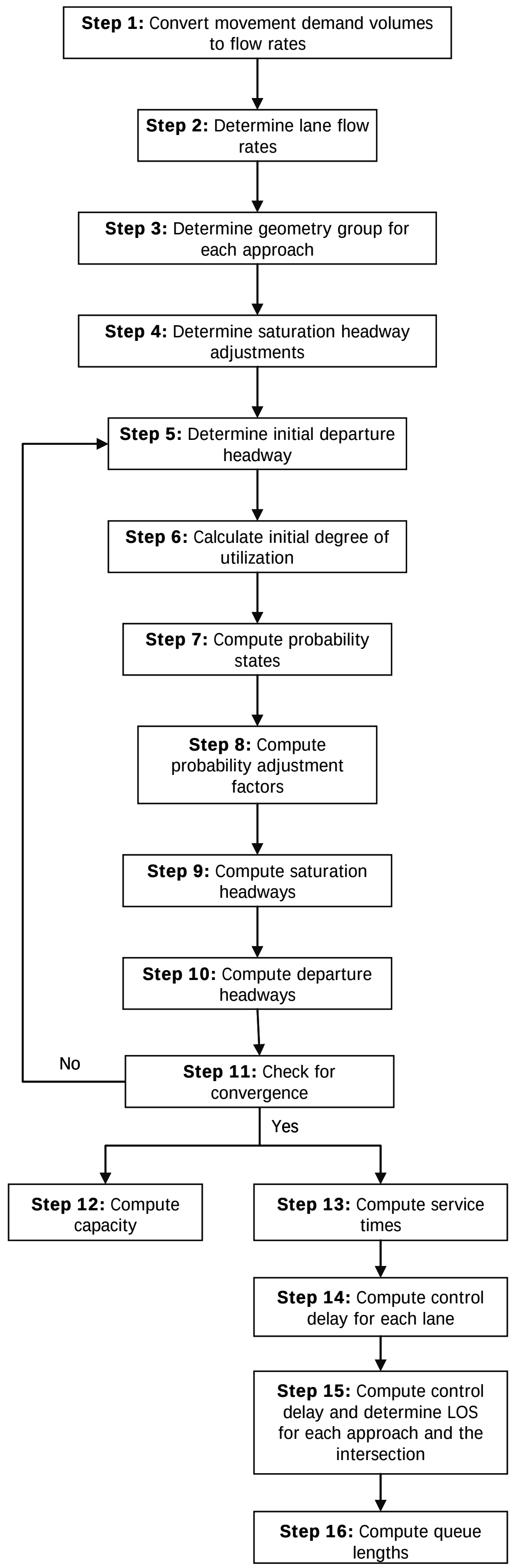
where

v_i = demand flow rate for movement i (veh/h),

V_i = demand volume for movement i (veh/h), and

PHF = peak hour factor.

Exhibit 20-9
AWSC Intersection Methodology



Step 2: Determine Lane Flow Rates

For multilane approaches, the flow rate for each lane by movement is determined. If a certain movement can use more than one lane and its traffic volume distribution per lane is unknown, an equal distribution of volume among the lanes can be assumed.

Step 3: Determine Geometry Group for Each Approach

Exhibit 20-10 is consulted to determine the geometry group for each approach. The geometry group is needed to look up base saturation headways and headway adjustment factors.

Exhibit 20-10
Geometry Groups

Intersection Configuration	Number of Lanes: Subject Approach	Number of Lanes: Opposing Approach	Number of Lanes: Conflicting Approaches[a]	Geometry Group
Four leg or T	1	0 or 1	1	1
Four leg or T	1	0 or 1	2	2
Four leg or T	1	2	1	3a/4a
T	1	2	2	3b
Four leg	1	2	2	4b
Four leg or T	1	0 or 1	3	5
	1	3	1	
	2	0, 1, or 2	1 or 2	
	3	0 or 1	1	
	3	0 or 1	2 or 3	
	3	2 or 3	1	
Four leg or T	1	3	2	6
	1	2	3	
	1	3	3	
	2	3	1, 2, or 3	
	2	0, 1, 2 or 3	3	
	3	2 or 3	2 or 3	

Note: [a] If the number of lanes on conflicting approaches is different, the higher of the two should be used.

Step 4: Determine Saturation Headway Adjustments

The headway adjustment for each lane is computed by Equation 20-13. Saturation headway adjustments for left turns, right turns, and heavy vehicles are given in Exhibit 20-11.

Equation 20-13

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

where

h_{adj} = headway adjustment (s),

$h_{LT,adj}$ = headway adjustment for left turns (see Exhibit 20-11) (s),

$h_{RT,adj}$ = headway adjustment for right turns (see Exhibit 20-11) (s),

$h_{HV,adj}$ = headway adjustment for heavy vehicles (see Exhibit 20-11) (s),

P_{LT} = proportion of left-turning vehicles in the lane,

P_{RT} = proportion of right-turning vehicles in the lane, and

P_{HV} = proportion of heavy vehicles in the lane.

Exhibit 20-11 Saturation Headway Adjustments by Geometry Group

Factor	Saturation Headway Adjustment (s)							
	Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
LT	0.2	0.2	0.2	0.2	0.2	0.2	0.5	0.5
RT	–0.6	–0.6	–0.6	–0.6	–0.6	–0.6	–0.7	–0.7
HV	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

Note: LT = left turns, RT = right turns, HV = heavy vehicles.

Step 5: Determine Initial Departure Headway

The process of determining departure headways (and thus service times) for each of the lanes on each of the approaches is iterative. For the first iteration, an initial departure headway of 3.2 s should be assumed. For subsequent iterations, the calculated values of departure headway from the previous iteration should be used if the calculation has not converged (see Step 11).

Step 6: Calculate Initial Degree of Utilization

By using the lane flow rates from Step 2 and the assumed initial departure headway from Step 5, the initial degree of utilization, x, is computed with Equation 20-14. If it is not the final iteration, and the degree of utilization exceeds 1, then the degree of utilization should be reset to 1.

Equation 20-14

$$x = \frac{v h_d}{3{,}600}$$

Step 7: Compute Probability States

The probability state of each combination i is found with Equation 20-15.

Equation 20-15

$$P(i) = \prod_j P(a_j)$$

where

j = O1 (opposing approach, Lane 1), O2 (opposing approach, Lane 2), CL1 (conflicting left approach, Lane 1), CL2 (conflicting left approach, Lane 2), CR1 (conflicting right approach, Lane 1), and CR2 (conflicting right approach, Lane 2) for a two-lane, two-way AWSC intersection;

$P(a_j)$ = probability of a_j, computed on the basis of Exhibit 20-12, where V_j is the lane flow rate; and

a_j = 1 (indicating a vehicle present) or 0 (indicating no vehicle present in the lane) (values of a_j for each lane in each combination i are listed in Exhibit 20-13).

Exhibit 20-12 Probability of a_j

a_j	V_j	$P(a_j)$
1	0	0
0	0	1
1	> 0	x_j
0	> 0	$1 - x_j$

Note: x is the degree of utilization defined in Equation 20-14.

Tables for three-lane approaches are given in Chapter 32, STOP-Controlled Intersections: Supplemental

Exhibit 20-13 provides the 64 possible combinations when alternative lane occupancies are considered for two-lane approaches. A 1 indicates that a vehicle is in the lane, and a 0 indicates that a vehicle is not in the lane. A similar table for three lanes on each approach is provided in Chapter 32 in Volume 4.

Exhibit 20-13
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Two-Lane Approaches, by Lane)

i	DOC Case	Number of Vehicles	Opposing Approach L1	Opposing Approach L2	Conflicting Left Approach L1	Conflicting Left Approach L2	Conflicting Right Approach L1	Conflicting Right Approach L2
1	1	0	0	0	0	0	0	0
2	2	1	1	0	0	0	0	0
3			0	1	0	0	0	0
4		2	1	1	0	0	0	0
5	3	1	0	0	1	0	0	0
6			0	0	0	1	0	0
7			0	0	0	0	1	0
8			0	0	0	0	0	1
9		2	0	0	1	1	0	0
10			0	0	0	0	1	1
11	4	2	0	0	0	1	0	1
12			0	0	1	0	0	1
13			0	0	1	0	1	0
14			0	0	0	1	1	0
15			0	1	0	1	0	0
16			1	0	1	0	0	0
17			0	1	0	0	1	0
18			1	0	0	1	0	0
19			0	1	1	0	0	0
20			0	1	0	0	0	1
21			1	0	0	0	1	0
22			1	0	0	0	0	1
23		3	0	0	0	1	1	1
24			0	0	1	1	0	1
25			0	0	1	1	1	0
26			1	0	1	1	0	0
27			1	1	1	0	0	0
28			1	1	0	0	1	0
29			1	1	0	0	0	1
30			0	1	1	1	0	0
31			1	0	0	0	1	1
32			0	0	1	0	1	1
33			1	1	0	1	0	0
34			0	1	0	0	1	1
35		4	1	1	0	0	1	1
36			0	0	1	1	1	1
37			1	1	1	1	0	0
38	5	3	0	1	0	1	0	1
39			1	0	0	1	1	0
40			0	1	1	0	1	0
41			0	1	0	1	1	0
42			0	1	1	0	0	1
43			1	0	1	0	0	1
44			1	0	0	1	0	1
45			1	0	1	0	1	0
46		4	1	0	0	1	1	1
47			0	1	1	1	1	0
48			0	1	1	1	0	1
49			1	0	1	0	1	1
50			1	0	1	1	1	0
51			0	1	0	1	1	1
52			1	1	1	0	0	1
53			1	0	1	1	0	1
54			0	1	1	0	1	1
55			1	1	0	1	1	0
56			1	1	0	1	0	1
57			1	1	1	0	1	0
58		5	1	0	1	1	1	1
59			1	1	0	1	1	1
60			1	1	1	0	1	1
61			0	1	1	1	1	1
62			1	1	1	1	1	0
63			1	1	1	1	0	1
64		6	1	1	1	1	1	1

Note: DOC case is the degree-of-conflict case, number of vehicles is the total number of vehicles on the opposing and conflicting approaches, L1 is Lane 1, and L2 is Lane 2.

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 20-16 through Equation 20-20 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined (assuming the 64 cases presented in Exhibit 20-13).

Equation 20-16
$$P(C_1) = P(1)$$

Equation 20-17
$$P(C_2) = \sum_{i=2}^{4} P(i)$$

Equation 20-18
$$P(C_3) = \sum_{i=5}^{10} P(i)$$

Equation 20-19
$$P(C_4) = \sum_{i=11}^{37} P(i)$$

Equation 20-20
$$P(C_5) = \sum_{i=38}^{64} P(i)$$

The probability adjustment factors are then computed with Equation 20-21 through Equation 20-25.

Equation 20-21
$$AdjP(1) = \alpha\left[P(C_2) + 2P(C_3) + 3P(C_4) + 4P(C_5)\right]/1$$

Equation 20-22
$$AdjP(2) \text{ through } AdjP(4) = \alpha\left[P(C_3) + 2P(C_4) + 3P(C_5) - P(C_2)\right]/3$$

Equation 20-23
$$AdjP(5) \text{ through } AdjP(10) = \alpha\left[P(C_4) + 2P(C_5) - 3P(C_3)\right]/6$$

Equation 20-24
$$AdjP(11) \text{ through } AdjP(37) = \alpha\left[P(C_5) - 6P(C_4)\right]/27$$

Equation 20-25
$$AdjP(38) \text{ through } AdjP(64) = -\alpha\left[10P(C_5)\right]/27$$

where α equals 0.01 (or 0.00 if correlation among saturation headways is not taken into account).

The adjusted probability $P'(i)$ for each combination is simply the sum of $P(i)$ and $AdjP(i)$, as given by Equation 20-26.

Equation 20-26
$$P'(i) = P(i) + AdjP(i)$$

Step 9: Compute Saturation Headways

The saturation headway h_{si} is the sum of the base saturation headway as presented in Exhibit 20-14 and the saturation headway adjustment factor from Step 4. It is shown in Equation 20-27.

Equation 20-27
$$h_{si} = h_{base} + h_{adj}$$

Exhibit 20-14
Saturation Headway Values by Case and Geometry Group

Case	No. of Veh.	Base Saturation Headway (s) Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
1	0	3.9	3.9	4.0	4.3	4.0	4.5	4.5	4.5
2	1	4.7	4.7	4.8	5.1	4.8	5.3	5.0	6.0
	2							6.2	6.8
	≥3								7.4
3	1	5.8	5.8	5.9	6.2	5.9	6.4	6.4	6.6
	2							7.2	7.3
	≥3								7.8
4	2	7.0	7.0	7.1	7.4	7.1	7.6	7.6	8.1
	3							7.8	8.7
	4							9.0	9.6
	≥5								12.3
5	3	9.6	9.6	9.7	10.0	9.7	10.2	9.7	10.0
	4							9.7	11.1
	5							10.0	11.4
	≥6							11.5	13.3

Step 10: Compute Departure Headways

The departure headway of the approach is the expected value of the saturation headway distribution, given by Equation 20-28.

$$h_d = \sum_{i=1}^{64} P'(i) h_{si}$$

Equation 20-28

where i represents each combination of the five degree-of-conflict cases and h_{si} is the saturation headway for that combination.

Step 11: Check for Convergence

The calculated values of h_d are checked against the initial values assumed for h_d. If the values change by more than 0.1 s (or a more precise measure of convergence), Steps 5 through 10 are repeated until the values of departure headway for each lane do not change significantly.

Step 12: Compute Capacity

Capacity is estimated for a stated set of opposing and conflicting volumes.

The capacity of each approach is computed under the assumption that the flows on the opposing and conflicting approaches are constant. The given flow rate on the subject lane is increased and the departure headways are computed for each approach until the degree of utilization for the subject lane reaches 1. When this occurs, the final value of the subject approach flow rate is the maximum possible throughput or capacity of this lane.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time with Equation 20-29.

$$t_s = h_d - m$$

Equation 20-29

where t_s is the service time, h_d is the departure headway, and m is the move-up time (2.0 s for Geometry Groups 1 through 4; 2.3 s for Geometry Groups 5 and 6).

Step 14: Compute Control Delay for Each Lane

The delay experienced by a motorist is made up of a number of factors that relate to control, geometrics, traffic, and incidents. Control delay is the difference between the travel time that is actually experienced and the reference travel time that would result during conditions in the absence of traffic control or conflicting traffic.

Equation 20-30 can be used to compute control delay for each lane.

Equation 20-30

$$d = t_s + 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{h_d x}{450T}}\right] + 5$$

where

d = average control delay (s/veh),

x = $vh_d/3{,}600$ = degree of utilization,

t_s = service time (s),

h_d = departure headway (s), and

T = length of analysis period (h).

Step 15: Compute Control Delay and Determine LOS for Each Approach and the Intersection

The control delay for an approach is calculated by computing a weighted average of the delay for each lane on the approach, weighted by the volume in each lane. The calculation is shown in Equation 20-31.

Equation 20-31

$$d_{\text{approach}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

d_{approach} = control delay for the approach (s/veh),

d_i = control delay for lane i (s/veh), and

v_i = flow rate for lane i (veh/h).

The control delay for the intersection as a whole is similarly calculated by computing a weighted average of the delay for each approach, weighted by the volume on each approach. It is shown in Equation 20-32.

Equation 20-32

$$d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

$d_{\text{intersection}}$ = control delay for the entire intersection (s/veh),

d_i = control delay for approach i (s/veh), and

v_i = flow rate for approach i (veh/h).

The LOS for each approach and for the intersection are determined with Exhibit 20-2 and the computed values of control delay.

Step 16: Compute Queue Lengths

Research (3) has determined that the methodology for predicting queues at TWSC intersections can be applied to AWSC intersections. As such, the mean queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest.

Equation 20-33 can be used to calculate the 95th percentile queue for each approach lane.

$$Q_{95} \approx \frac{900T}{h_d}\left[(x-1)+\sqrt{(x-1)^2+\frac{h_d x}{150T}}\right]$$

Equation 20-33

where

Q_{95} = 95th percentile queue (veh),

x = $vh_d/3{,}600$ = degree of utilization,

h_d = departure headway (s), and

T = length of analysis period (h).

PEDESTRIAN MODE

Data collection and research are needed to determine an appropriate LOS methodology for pedestrians at AWSC intersections.

Applying the LOS procedures used to determine pedestrian delay at TWSC intersections to AWSC intersections does not produce intuitive or usable results. The TWSC delay calculations apply only for crossings where conflicting traffic is not STOP-controlled (i.e., pedestrians crossing the major street at a TWSC intersection). Approaches where conflicting traffic is STOP-controlled (i.e., pedestrians crossing the minor street at a TWSC intersection) are assumed to result in negligible delay for pedestrians, as vehicles are required to stop and wait for conflicting vehicle and pedestrian traffic before proceeding.

As such, applying the TWSC methodology to pedestrians at AWSC intersections results in negligible delay for all pedestrians at all approaches. The reality of AWSC intersection operations for pedestrians is much different, however, and generally results in at least some delay for pedestrians. The amount of delay incurred will depend on a number of operating and geometric characteristics of the intersection in question. While no quantitative methodology accounting for these factors is available, several of the most important factors are discussed qualitatively below.

The operational characteristics of AWSC intersections for pedestrians largely depend on driver behavior. In most cases, drivers are legally required to yield to pedestrians crossing or preparing to cross AWSC intersections. However, it should be expected that operations differ significantly depending on enforcement levels, region of the country, and location (e.g., urban, suburban, or rural).

Traffic Volumes

At intersections with relatively low traffic volumes, there are generally no standing queues of vehicles at AWSC approaches. In these cases, pedestrians attempting to cross an approach of the intersection will typically experience little

or no delay, as they will be able to proceed almost immediately after reaching the intersection.

At AWSC intersections with higher volumes, there are typically standing queues of motor vehicles on each approach. These intersections operate in a two-phase or four-phase sequence, as described earlier and depicted in Exhibit 20-3. In these situations, the arrival of a pedestrian does not typically disrupt the normal phase operations of the intersection. Rather, the pedestrian is often forced to wait until the phase arrives for vehicles in the approach moving adjacent to the pedestrian.

Under a scenario in which the intersection functions under the operations described above for pedestrians, average pedestrian delay might be expected to be half of the time needed to cycle through all phases for the particular intersection, assuming random arrival of pedestrians. However, several other factors may also affect pedestrian delay and operations at AWSC intersections, as described below.

Number of Approach Lanes

As the number of approach lanes at AWSC intersections increases, pedestrian crossing distance increases proportionally, resulting in significantly longer pedestrian crossing times compared with single-lane intersections. In addition, vehicles already in the intersection or about to enter the intersection take longer to complete their movement. As a result, pedestrians at multilane AWSC intersections may wait longer before taking their turn to cross.

Proportion of Turning Traffic

The ability of a pedestrian to cross at an AWSC intersection may also depend on the proportion of through motor vehicle traffic to turning motor vehicle traffic. As described above, pedestrians may often cross during the phase in which adjacent motor vehicle traffic traverses the intersection. However, when an adjacent motor vehicle is turning, that vehicle will conflict with pedestrians attempting to cross. Because of the additional conflicts with pedestrians created by turning vehicles at AWSC intersections, pedestrian delay may be expected to rise as the proportion of turning vehicles increases, similar to the effect that turning proportion has on vehicular delay.

Pedestrian Volumes

Under most circumstances, there is adequate capacity for all pedestrians queued for a given movement at an AWSC intersection to cross simultaneously with adjacent motor vehicle traffic. However, in locations with very high pedestrian volumes, this may not be the case. The total pedestrian capacity of a particular AWSC intersection phase is limited by both the width of the crosswalk (how many pedestrians can cross simultaneously) and driver behavior.

In situations in which not all queued pedestrians may cross during a particular phase, pedestrian delay will increase, as some pedestrians will be forced to wait through an additional cycle of intersection phases before crossing. However, pedestrian volumes in this range are unlikely to occur often; rather,

intersections with pedestrian volumes high enough to cause significant delay are typically signalized.

BICYCLE MODE

Where bicycles queue with motor vehicles on AWSC approaches, the procedures described to estimate motor vehicle delay can be applied to bicycles. However, bicycles differ from motor vehicles in that they do not queue linearly at STOP signs. Instead, multiple bicycles often cross at the same time as the adjacent vehicular traffic stream. This phenomenon has not been researched as of the time of publication of this edition of the HCM and is not explicitly included in the methodology.

Where an AWSC approach provides a bicycle lane, bicycle delay will be significantly different and, in general, lower than motor vehicle delay. The exception is bicycles intending to turn left; those cyclists will typically queue with motor vehicles. Where bicycle lanes are available, bicycles are able to move unimpeded until reaching the stop line, as the bike lane allows the cyclist to pass any queued motor vehicles on the right. In this situation, bicycles will still incur delay upon reaching the intersection.

In most cases, bicycles will be able to travel through the intersection concurrently with adjacent motor vehicle traffic. This, in effect, results in multilane operations, with the bike lane serving as the curb lane, meaning that bicycles will be delayed from the time of arrival at the intersection until the adjacent motor vehicle phase occurs. As noted above, multiple bicycles will likely be able to cross simultaneously through the intersection. Finally, even where bicycle lanes are not available, many cyclists still pass queued motor vehicles on the right, resulting in lower effective bicycle delay compared with motor vehicle delay.

3. APPLICATIONS

DEFAULT VALUES

A comprehensive presentation of potential default values for interrupted-flow facilities is available elsewhere (*4*). These defaults cover the key characteristics of peak hour factor (*PHF*) and percent heavy vehicles (%HV). Recommendations are based on geographic region, population, and time of day. All general default values for interrupted flow facilities may be applied to analysis of AWSC intersections in the absence of field data or projections of conditions.

Both demand volumes and the number and configuration of lanes at the intersection are site specific and do not lend themselves to default values. The following default values may be applied to an AWSC intersection analysis:

- Peak hour factor (*PHF*) = 0.92
- Percent heavy vehicles (%HV) = 3%

As the number of default values used in any analysis increases, the accuracy of the result becomes more approximate and may differ significantly from the actual outcome, depending on local conditions.

ESTABLISH INTERSECTION ANALYSIS BOUNDARIES

This methodology assumes that the AWSC intersection under investigation is isolated. When interaction effects are likely between the subject AWSC intersection and other intersections (e.g., queue spillback, demand starvation), the use of alternative tools may result in a more accurate analysis. Analysis boundaries may also include different demand scenarios related to time of day or to different development scenarios that produce different demand flow rates.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design analysis.

Operational Analysis

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement (veh/h), heavy vehicle percentages for each approach, peak hour factor for all demand volumes, and lane configuration. The outputs of an operational analysis are estimates of capacity and control delay. The steps of the methodology, described in the Methodology section, are followed directly without modification.

Design Analysis

The operational analysis described earlier in this chapter can be used for design purposes by using a given set of traffic flow data and iteratively determining the number and configuration of lanes that would be required to produce a given LOS.

The operational analysis methodology for AWSC intersections can also be used for design analysis and planning and preliminary engineering analysis.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of an AWSC intersection. To estimate LOS for a future time horizon, a planning analysis based on the operational method is used. The planning method uses all the geometric and traffic flow data required for an operational analysis, and the computations are identical. However, input variables for heavy-vehicle percentage and peak hour factor are typically estimated (or defaults used) when planning applications are performed.

USE OF ALTERNATIVE TOOLS

Except for the effects of interaction with other intersections, the limitations of the methodology that were stated earlier in this chapter have minimal potential to be addressed by alternative tools. Therefore, insufficient experience with alternative tools is available as of the time of publication of this edition of the HCM to support the development of useful guidance for their application to AWSC intersections.

4. EXAMPLE PROBLEM

An additional AWSC example problem is provided in Chapter 32, STOP-Controlled Intersections: Supplemental.

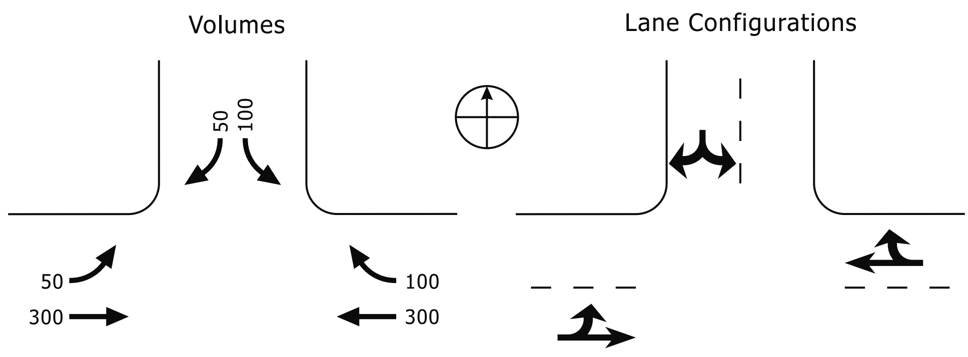
EXAMPLE PROBLEM 1: SINGLE-LANE, T-INTERSECTION

The Facts

The following describes this location's traffic and geometric characteristics:

- Three legs (T-intersection),
- One-lane entries on each leg,
- Percent heavy vehicles on all approaches = 2%,
- Peak hour factor = 0.95, and
- Volumes and lane configurations as shown below (Exhibit 20-15).

Exhibit 20-15
Volumes and Lane Configurations for Example Problem 1



Comments

The use of a spreadsheet or software for AWSC intersection analysis is recommended because of the repetitive and iterative computations required.

All input parameters are known, so no default values are needed or used. The use of a spreadsheet or software is recommended because of the repetitive computations required. Slight differences in reported values may result from rounding differences between manual and software computations.

Step 1: Convert Movement Demand Volumes to Flow Rates

Peak 15-min flow rates for each turning movement at the intersection are equal to the hourly volumes divided by *PHF*. For example, the peak 15-min flow rate for the eastbound through movement is as follows:

$$v_{EBTH} = \frac{V_{EBTH}}{PHF} = \frac{300}{0.95} = 316 \text{ veh/h}$$

Step 2: Determine Lane Flow Rates

This step does not apply because the intersection has one-lane approaches on all legs.

Step 3: Determine Geometry Group for Each Approach

Exhibit 20-10 shows that each approach should be assigned to Geometry Group 1.

Step 4: Determine Saturation Headway Adjustments

Exhibit 20-11 shows that the headway adjustments for left turns, right turns, and heavy vehicles are 0.2, –0.6, and 1.7, respectively. These values apply to all approaches because all are assigned Geometry Group 1. Therefore, the saturation headway adjustment for the eastbound approach is as follows:

$$h_{adj} = h_{LT,adj} P_{LT} + h_{RT,adj} P_{RT} + h_{HV,adj} P_{HV}$$

$$h_{adj} = 0.2 \frac{53}{53+316} - 0.6(0) + 1.7(0.02) = 0.063$$

Similarly, the saturation headway adjustment for the westbound approach is as follows:

$$h_{adj} = 0.2(0) - 0.6 \frac{105}{105+316} + 1.7(0.02) = -0.116$$

Finally, the saturation headway adjustment for the southbound approach is as follows:

$$h_{adj} = 0.2 \frac{105}{105+53} - 0.6 \frac{53}{105+53} + 1.7(0.02) = -0.034$$

Steps 5 Through 11: Determine Departure Headway

These steps are iterative. The following narrative highlights some of the key calculations using the eastbound approach for Iteration 1 but does not attempt to reproduce all calculations for all iterations. Full documentation of the example problem is included in Chapter 32, STOP-Controlled Intersections: Supplemental.

Step 6: Calculate Initial Degree of Utilization

By using the lane flow rates from Step 2 and the assumed initial departure headway from Step 5, the initial degree of utilization, x, for the eastbound approach is computed as follows:

$$x, EB = \frac{vh_d}{3{,}600} = \frac{(368)(3.2)}{3{,}600} = 0.327$$

$$x, WB = \frac{(421)(3.2)}{3{,}600} = 0.374$$

$$x, NB = \frac{vh_d}{3{,}600} = \frac{(158)(3.2)}{3{,}600} = 0.140$$

Step 7: Compute Probability States

The probability state of each combination i is determined with Equation 20-15.

$$P(i) = \prod_j P(a_j) = P(a_O)P(a_{CL})P(a_{CR})$$

For an intersection with single-lane approaches, only these eight cases from Exhibit 20-13 apply:

i	DOC Case	Number of Vehicles	Opposing Approach	Conflicting Left Approach	Conflicting Right Approach
1	1	0	0	0	0
2	2	1	1	0	0
5	3	1	0	1	0
7	3	1	0	0	1
13	4	2	0	1	1
16	4	2	1	1	0
21	4	2	1	0	1
45	5	3	1	1	1

For example, the probability state for the eastbound leg under the condition of no opposing vehicles on the other approaches (degree of conflict Case 1, $i = 1$) is as follows (using Exhibit 20-6):

$$P(a_O) = 1 - x_O = 1 - 0.374 = 0.626 \quad \text{(no opposing present)}$$

$$P(a_{CL}) = 1 - x_{CL} = 1 - 0.140 = 0.860 \quad \text{(no conflicting from left present)}$$

$$P(a_{CR}) = 1 \quad \text{(no approach conflicting from right)}$$

Therefore,

$$P(1) = P(a_O)P(a_{CL})P(a_{CR}) = (0.626)(0.860)(1) = 0.538$$

Similarly,

$$P(2) = (0.374)(0.860)(1) = 0.322$$

$$P(5) = (0.626)(0.140)(1) = 0.088$$

$$P(7) = (0.626)(0.860)(0) = 0$$

$$P(13) = (0.626)(0.140)(0) = 0$$

$$P(16) = (0.374)(0.140)(1) = 0.052$$

$$P(21) = (0.374)(0.860)(0) = 0$$

$$P(45) = (0.374)(0.140)(0) = 0$$

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed as follows:

$$P(C_1) = P(1) = 0.538$$

$$P(C_2) = P(2) = 0.322$$

$$P(C_3) = P(5) + P(7) = 0.088 + 0 = 0.088$$

$$P(C_4) = P(13) + P(16) + P(21) = 0 + 0.052 + 0 = 0.052$$

$$P(C_5) = P(45) = 0$$

The probability adjustment factors for the nonzero cases are as follows:

$$AdjP(1) = 0.01[0.322 + 2(0.088) + 3(0.052) + 0]/1 = 0.0065$$

$$AdjP(2) = 0.01[(0.088) + 2(0.052) + 0 - 0.322]/3 = -0.0004$$

$$AdjP(5) = 0.01[(0.052) + 2(0) - 3(0.088)]/6 = -0.0004$$

$$AdjP(16) = 0.01[0 - 6(0.052)]/27 = -0.0001$$

Therefore, the adjusted probability for Combination 1, for example, is as follows:

$$P'(1) = 0.538 + 0.0065 = 0.5445$$

Step 9: Compute Saturation Headways

The base saturation headways for each combination can be determined with Exhibit 20-14. They are adjusted by using the adjustment factors calculated in Step 4 and added to the base saturation headways to determine saturation headways as follows (eastbound illustrated):

i	h_{base}	h_{adj}	h_{si}
1	3.9	0.063	3.963
2	4.7	0.063	4.763
5	5.8	0.063	5.863
7	7.0	0.063	7.063

Step 10: Compute Departure Headways

The departure headway of the approach is the sum of the products of the adjusted probabilities and the saturation headways as follows (eastbound illustrated):

$$h_d = (0.5445)(3.963) + (0.3213)(4.763) + (0.0875)(5.863) + (0.0524)(7.063) = 4.57$$

Step 11: Check for Convergence

The calculated values of h_d are checked against the initial values assumed for h_d. After one iteration, each calculated headway differs from the initial value by more than 0.1 s. Therefore, the new calculated headway values are used as initial values in a second iteration. For this problem, four iterations are required for convergence.

	EB L1	EB L2	WB L1	WB L2	NB L1	NB L2	SB L1	SB L2
Total Lane Flow Rate	368		421				158	
hd, initial value, iteration 1	3.2		3.2				3.2	
x, initial, iteration 1	0.327		0.374				0.140	
hd, computed value, iteration 1	4.57		4.35				5.14	
Convergence?	N		N				N	
hd, initial value, iteration 2	4.57		4.35				5.14	
x, initial, iteration 2	0.468		0.509				0.225	
hd, computed value, iteration 2	4.88		4.66				5.59	
Convergence?	N		N				N	
hd, initial value, iteration 3	4.88		4.66				5.59	
x, initial, iteration 3	0.499		0.545				0.245	
hd, computed value, iteration 3	4.95		4.73				5.70	
Convergence?	Y		Y				N	
hd, initial value, iteration 4	4.88		4.66				5.70	
x, initial, iteration 4	0.499		0.545				0.250	
hd, computed value, iteration 4	4.97		4.74				5.70	
Convergence?	Y		Y				Y	

Step 12: Compute Capacity

The capacity of each approach is computed by increasing the given flow rate on the subject lane (assuming the flows on the opposing and conflicting approaches are constant) until the degree of utilization for the subject lane reaches 1. This level of calculation requires running an iterative procedure many times, which is practical for a spreadsheet or software implementation.

Here, the eastbound approach capacity is approximately 720 veh/h, which is lower than the value that could be estimated by dividing the approach volume by the degree of utilization (368/0.492 = 748 veh/h). The difference is due to the interaction effects among the approaches: increases in eastbound traffic volume increase the departure headways of the other approaches, which in turn increases the departure headway of the subject approach.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time by using Equation 20-29. For the eastbound approach (using a value for m of 2.0 for Geometry Group 1), the calculation is as follows:

$$t_s = h_d - m = 4.97 - 2.0 = 2.97$$

Step 14: Compute Control Delay

The control delay for each approach is computed with Equation 20-30 as follows (eastbound approach illustrated):

$$d = 2.97 + 900(0.25)\left[(0.508-1)+\sqrt{(0.508-1)^2+\frac{4.97(0.508)}{450(0.25)}}\right]+5 = 13.0\text{ s}$$

By using Exhibit 20-2, the eastbound approach is assigned LOS B. A similar calculation for the westbound and southbound approaches yields 13.5 and 10.6 s, respectively.

The control delays for the approaches can be combined into an intersection control delay by using a weighted average as follows:

$$d = \frac{(13.0)(368)+(13.5)(421)+(10.6)(158)}{368+421+158} = 12.8\text{ s}$$

This value of delay is assigned LOS B.

Step 15: Compute Queue Length

The 95th percentile queue for each lane is computed with Equation 20-33 as follows (eastbound approach illustrated):

$$Q_{95} \approx \frac{900(0.25)}{4.97}\left[(0.508-1)+\sqrt{(0.508-1)^2+\frac{4.97(0.508)}{150(0.25)}}\right] = 2.9\text{ veh}$$

This queue length would be reported as 3 vehicles.

Discussion

The results indicate that the intersection operates well with low delays.

5. REFERENCES

These references are available in the Technical Reference Library in Volume 4.

1. Kyte, M., Z. Tian, Z. Mir, Z. Hameedmansoor, W. Kittelson, M. Vandehey, B. Robinson, W. Brilon, L. Bondzio, N. Wu, and R. Troutbeck. *NCHRP Web Document 6: Capacity and Level of Service at Unsignalized Intersections: Final Report, Vol. 2—All-Way Stop-Controlled Intersections*. Transportation Research Board, National Research Council, Washington, D.C., April 1996.
2. Richardson, A. J. A Delay Model for Multiway Stop-Sign Intersections. In *Transportation Research Record 1112*, Transportation Research Board, National Research Council, Washington, D.C., 1987, pp. 107–114.
3. Tian, Z., and M. Kyte. Queue Length Models for All-Way Stop-Controlled Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1988*, Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 63–66.
4. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses*. Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 21
ROUNDABOUTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Roundabouts are intersections with a generally circular shape, characterized by yield on entry and circulation around a central island (counterclockwise in the United States). Roundabouts have been used successfully throughout the world and are being used increasingly in the United States, especially since 1990.

Chapter 21, Roundabouts, presents concepts and procedures for analyzing these intersections. National Cooperative Highway Research Program Project 3-65 (*1*) provided a comprehensive database of roundabout operations for U.S. conditions on the basis of a study of 31 sites. The procedures that follow are largely founded on that study's recommendations. These procedures allow the analyst to assess the operational performance of an existing or planned one-lane or two-lane roundabout given traffic demand levels.

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The analytical procedure presented in this chapter assumes that the analysis boundaries are the roundabout itself, including associated pedestrian crosswalks. Alternative tools discussed in this chapter can, in some cases, expand the analysis boundaries to include adjacent intersections. The methodology presented here includes discussion of motor vehicles, pedestrians, and bicycles.

LEVEL OF SERVICE CRITERIA

The level of service (LOS) criteria for automobiles in roundabouts are given in Exhibit 21-1. As the table notes, LOS F is assigned if the volume-to-capacity ratio of a lane exceeds 1.0 regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

The thresholds in Exhibit 21-1 are based on the considered judgment of the Transportation Research Board Committee on Highway Capacity and Quality of Service. As discussed later in this chapter, roundabouts share the same basic control delay formulation with two-way and all-way STOP-controlled intersections, adjusting for the effect of YIELD control. However, at the time of publication of this edition of the *Highway Capacity Manual* (HCM), no research was available on traveler perception of quality of service at roundabouts. In the absence of such research, the service measure and thresholds have been made consistent with those for other unsignalized intersections, primarily on the basis of this similar control delay formulation.

Exhibit 21-1
LOS Criteria: Automobile Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a]	
	v/c ≤ 1.0	v/c >1.0
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: [a] For approaches and intersectionwide assessment, LOS is defined solely by control delay.

REQUIRED INPUT DATA

The following data are required to analyze a roundabout:

1. Number and configuration of lanes on each approach;
2. Either of the following:
   a. Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak 15 min, or
   b. Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak hour, and a peak hour factor for the hour;
3. Percentage of heavy vehicles;
4. Volume distribution across lanes for multilane entries; and
5. Length of analysis period, generally a peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

SCOPE OF THE METHODOLOGY

The methodology presented in this chapter focuses on the operation of roundabouts. The methodology does not account for the effects of adjacent traffic control devices such as nearby traffic signals or signalized pedestrian crossings. This version of the roundabout analysis procedures results primarily from studies conducted by National Cooperative Highway Research Program Project 3-65 (*1*). The chapter also includes a discussion of alternative tools that can model situations beyond the scope of the analytical methodology presented in this chapter.

The methodology does not necessarily apply to other types of circular intersections such as rotaries, neighborhood traffic circles, or signalized traffic circles, because these types of circular intersections usually have geometric or traffic control elements that deviate from those used in roundabouts. As a result, their operational performance may be significantly different from that experienced at roundabouts and thus cannot be accurately modeled by using the procedures in this chapter. More detail on the differentiation between roundabouts and other circular intersections can be found elsewhere (*2*, *3*).

LIMITATIONS OF THE METHODOLOGY

This chapter's methodology applies to isolated roundabouts with up to two entry lanes and up to one bypass lane per approach.

While the database on which these procedures are based is the most comprehensive developed for U.S. conditions, it does not cover all situations that may be encountered in practice. The chapter's methodology applies to isolated roundabouts with up to two entry lanes and up to one bypass lane per approach.

Automobile Mode

The methodology presented for automobiles covers typical roundabout facilities quite well, but it lacks examples of situations in which

- Upstream or downstream signals (including, but not limited to, pedestrian signals) significantly influence the performance of the roundabout;

- Priority reversal occurs, such as unusual forced entry conditions under extremely high flows;
- A high level of pedestrian or bicycle activity is present;
- The roundabout is in close proximity to one or more other roundabouts;
- More than two entry lanes are present on one or more approaches; or
- One or more entry lanes are of limited, or short, length (a flared design).

Priority reversal can occur when entering traffic dominates an entry, causing circulating traffic to yield.

A typical flared entry is one that widens from one approach lane to two entry lanes. Other flaring combinations, including flares of lane width, are possible.

Pedestrian Mode

Research on the operational performance of pedestrians at roundabouts is limited, in terms of the effect of both motor vehicles on pedestrians and pedestrians on motor vehicles. This chapter's methodologies include international models and analytical tools that have not been validated by research in the United States at the time of publication of this edition of the HCM. Additional research on pedestrian operations at roundabouts is needed to develop and refine procedures that adequately address these issues.

Bicycle Mode

Current methodologies to analyze LOS and delay at roundabouts only apply to bicycles in limited situations and have not been validated by research in the United States at the time of publication of this edition of the HCM. Additional research on bicycle behavior and operations at roundabouts is needed to develop procedures that adequately address these issues.

2. METHODOLOGY

OVERVIEW

This chapter presents procedures for analyzing roundabouts, introduces the unique characteristics of roundabout capacity, and presents terminology specific to roundabouts. For ease of reference, the following terms are defined:

v_e = entry flow rate,

v_c = conflicting flow rate, and

v_{ex} = exit flow rate.

Intersection analysis models generally fall into two categories. *Regression models* use field data to develop statistically derived relationships between geometric features and performance measures such as capacity and delay. *Analytical models* are based on traffic flow theory combined with the use of field measures of driver behavior, resulting in an analytic formulation of the relationship between those field measures and performance measures such as capacity and delay.

The procedure in this chapter uses a combination of regression and analytical models.

Both of these types of models are applicable to roundabouts. Gap-acceptance models are an example of an analytical model and are commonly applied for analyzing unsignalized intersections because they capture driver behavior characteristics directly and can be made site-specific by custom-tuning the values used for those parameters. However, simple gap-acceptance models may not capture all of the observed behavior, and more complex gap-acceptance models that account for limited priority or reverse priority are difficult to calibrate. Regression models are often used in these cases in which an understanding of driver behavior characteristics is incomplete. On the basis of recent analysis of U.S. field data, the procedure presented in this chapter incorporates a combination of simple, lane-based regression and gap-acceptance models for both single-lane and double-lane roundabouts.

CAPACITY CONCEPTS

The capacity of a roundabout approach is directly influenced by flow patterns. The three flows of interest, the entering flow, the circulating flow, and the exiting flow, are shown in Exhibit 21-2.

The capacity of a roundabout approach decreases as the circulating flow increases.

The capacity of an approach decreases as the conflicting flow increases. In general, the primary conflicting flow is the circulating flow that passes directly in front of the subject entry. While the circulating flow directly conflicts with the entry flow, the exiting flow may also affect a driver's decision to enter the roundabout. This phenomenon is similar to the effect of the right-turning stream approaching from the left side of a two-way STOP-controlled intersection. Until these drivers complete their exit maneuver or right turn, there may be some uncertainty in the mind of the driver at the yield or stop line about the intentions of the exiting or turning vehicle. However, even though it may have an influence in some cases, including this effect did not significantly improve the overall fit of the capacity models to the data (*1*) and therefore is not included in this chapter's models.

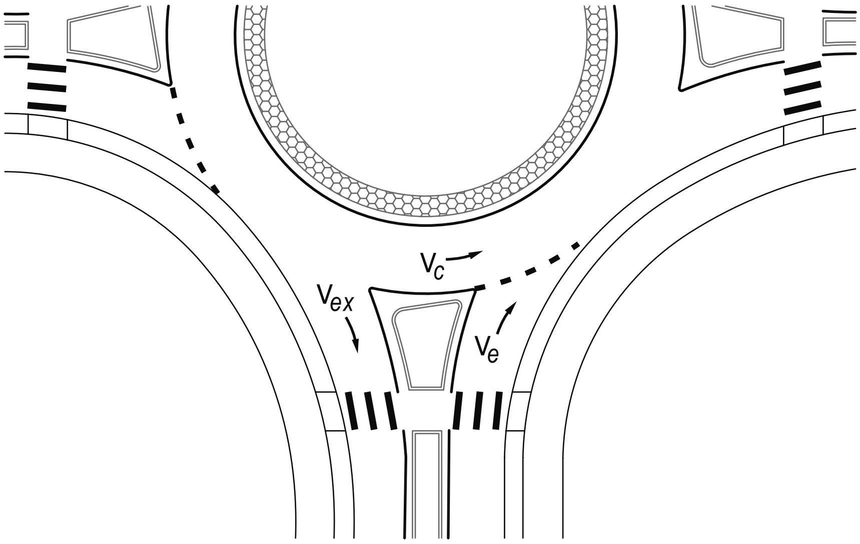


Exhibit 21-2
Analysis on One Roundabout Leg

When the conflicting flow rate approaches zero, the maximum entry flow is given by 3,600 s/h divided by the follow-up headway, which is analogous to the saturation flow rate for a movement receiving a green indication at a signalized intersection. At high levels of both entering and conflicting flow, limited priority (in which circulating traffic adjusts its headways to allow entering vehicles to enter), priority reversal (in which entering traffic forces circulating traffic to yield), and other behaviors may occur. In these cases, more complex analytical models or regression models, such as those incorporated into some of the alternative tools discussed later in this chapter, may give more realistic results.

When an approach operates over capacity during the analysis period, a condition known as *capacity constraint* may occur. During this condition, the actual circulating flow downstream of the constrained entry will be less than demand. The reduction in actual circulating flow may therefore increase the capacity of the affected downstream entries during this condition.

In addition, it has been suggested that origin–destination patterns have an influence on the capacity of a given entry (*4*, *5*). This effect was not identified in a recent study (*1*) and has not been incorporated into this chapter's models.

U.S. drivers presently seem to display more hesitation and caution in using roundabouts than drivers in other countries, which results in lower observed capacities. It is likely that capacities will increase in the future as U.S. drivers become more familiar with roundabouts.

Both roundabout design practices and the public's use of roundabouts are still maturing in the United States. Many of the sites that formed the database for this chapter were less than 5 years old when the data were collected. Although the available data were insufficient to definitively answer the question of whether capacity increases with driver familiarity, anecdotal observations suggest that this may well be the case. At this early stage of their introduction to roundabouts, American drivers seem to be displaying more hesitation and caution in the use of roundabouts than their international counterparts, which in turn has resulted in a lower observed capacity than might be ultimately achievable. It is therefore likely that capacity (and volumes) will increase in the years to come as more roundabouts are constructed in the United States and as user familiarity grows. Such an increase in capacity over time would be consistent with the historically observed trends in capacity for freeway facilities and signalized intersections, for example. On the other hand, capacities in the

United States over time may still be fundamentally different from those observed in other countries due to a variety of factors. These include limited use of turn indicators at roundabout exits by American drivers, differences in vehicle types, and the effect that the common use of STOP-controlled intersections (versus YIELD-controlled intersections) has had on drivers in the United States.

Single-Lane Roundabouts

The capacity of a single entry lane conflicted by one circulating lane (e.g., a single-lane roundabout, illustrated in Exhibit 21-3) is based on the conflicting flow. The equation for estimating the capacity is given as Equation 21-1:

Equation 21-1

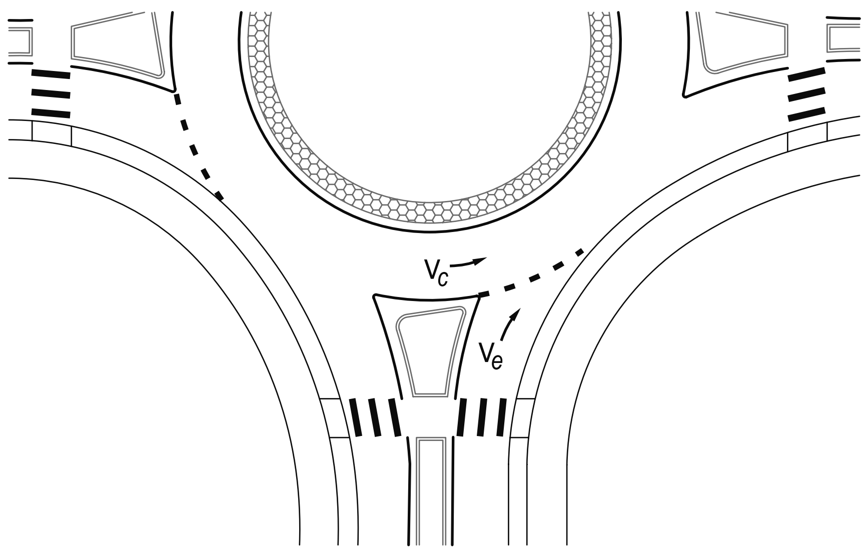
$$c_{e,pce} = 1{,}130 e^{\left(-1.0\times10^{-3}\right)v_{c,pce}}$$

where

$c_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and

$v_{c,pce}$ = conflicting flow rate (pc/h).

Exhibit 21-3
Example of One-Lane Entry Conflicted by One Circulating Lane



The capacity model given above reflects observations made at U.S. roundabouts in 2003. As noted previously, it is probable that U.S. roundabout capacity will increase to some degree with increased driver familiarity. In addition, communities with higher densities of roundabouts or generally more aggressive drivers may experience higher capacities. Therefore, local calibration of the capacity models is recommended to best reflect local driver behavior. This topic is discussed later in this chapter.

Multilane Roundabouts

Multilane roundabouts have more than one lane on at least one entry and at least part of the circulatory roadway. The number of entry, circulating, and exiting lanes may vary throughout the roundabout. Because of the many possible variations, the computational complexity is higher than for single-lane roundabouts.

The definition of headways and gaps for multilane facilities is more complicated than for single-lane facilities. If the circulating roadway truly functions as a multilane facility, then motorists at the entry perceive gaps in both the inside and outside lanes in some integrated fashion. Some drivers who choose to enter the roundabout via the right entry lane will yield to all traffic in the circulatory roadway due to their uncertainty about the path of the circulating vehicles. This uncertainty is more pronounced at roundabouts than at other unsignalized intersections due to the curvature of the circulatory roadway. However, some drivers in the right entry lane will enter next to a vehicle circulating in the inside lane if the circulating vehicle is not perceived to conflict. In addition, the behavior of circulating vehicles may be affected by the presence or absence of lane markings within the circulatory roadway. As a result, the gap-acceptance behavior of the right entry lane, in particular, is imperfect and difficult to quantify with a simple gap-acceptance model. This leads to an inclination toward using a regression-based model that implicitly accounts for these factors. More detail on the nuances of geometric design, pavement markings, and their relationship with operational performance can be found elsewhere (*2*, *3*).

For roundabouts with up to two circulating lanes, which is the only type of multilane roundabout addressed by the analytical methodology in this chapter, the entries and exits can be either one or two lanes wide (plus a possible right-turn bypass lane). The capacity model given above reflects observations made at a limited number of U.S. roundabouts in 2003. As discussed previously with single-lane roundabouts, local calibration of the capacity models (presented later in this chapter) is recommended to best reflect local driver behavior.

Capacity for Two-Lane Entries Conflicted by One Circulating Lane

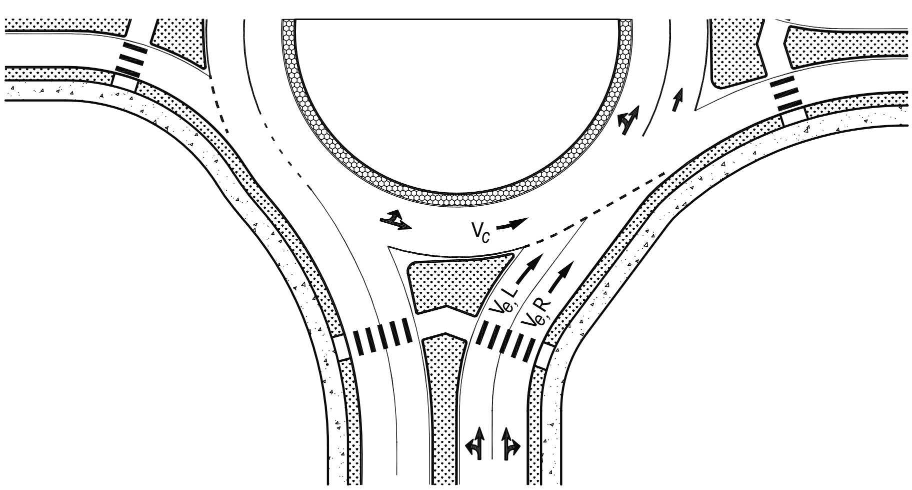
Equation 21-2 gives the capacity of each entry lane conflicted by one circulating lane (illustrated in Exhibit 21-4) as follows:

Equation 21-2

$$c_{e,pce} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce}}$$

where all variables are as defined previously.

Exhibit 21-4
Example of Two-Lane Entry Conflicted by One Circulating Lane



Capacity for One-Lane Entries Conflicted by Two Circulating Lanes

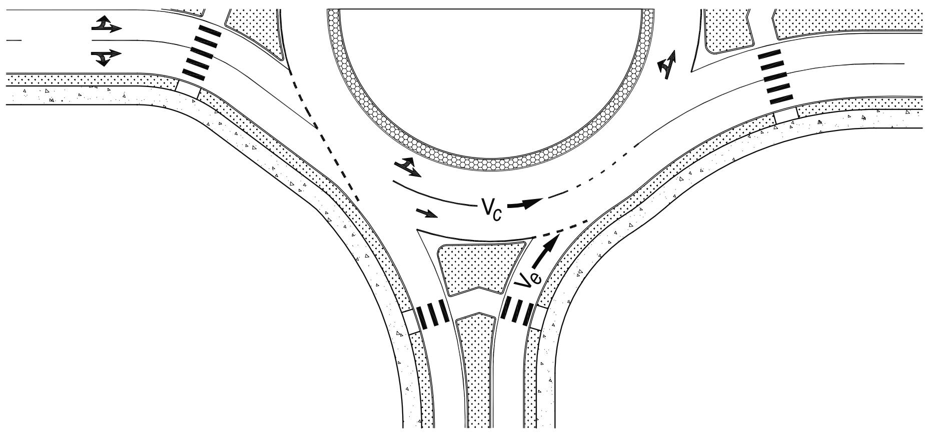
Equation 21-3 gives the capacity of a one-lane roundabout entry conflicted by two circulating lanes (illustrated in Exhibit 21-5) as follows:

Equation 21-3

$$c_{e,pce} = 1{,}130e^{\left(-0.7\times10^{-3}\right)v_{c,pce}}$$

where all variables are as defined previously ($v_{c,pce}$ is the total of both lanes).

Exhibit 21-5
Example of One-Lane Entry Conflicted by Two Circulating Lanes



Capacity for Two-Lane Entries Conflicted by Two Circulating Lanes

Equation 21-4 and Equation 21-5 give the capacity of the right and left lanes, respectively, of a two-lane roundabout entry conflicted by two circulating lanes (illustrated in Exhibit 21-6):

Equation 21-4

$$c_{e,R,pce} = 1{,}130e^{\left(-0.7\times10^{-3}\right)v_{c,pce}}$$

Equation 21-5

$$c_{e,L,pce} = 1{,}130e^{\left(-0.75\times10^{-3}\right)v_{c,pce}}$$

where

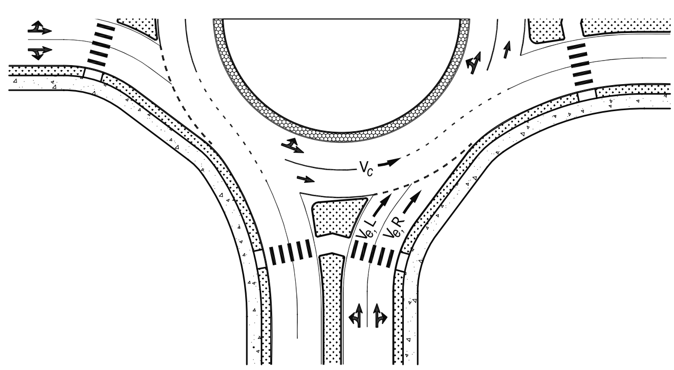
The capacity of the left lane of a roundabout approach is lower than the capacity of the right lane.

$c_{e,R,pce}$ = capacity of the right entry lane, adjusted for heavy vehicles (pc/h),

$c_{e,L,pce}$ = capacity of the left entry lane, adjusted for heavy vehicles (pc/h), and

$v_{c,pce}$ = conflicting flow rate (total of both lanes) (pc/h).

Exhibit 21-6
Example of Two-Lane Entry Conflicted by Two Circulating Lanes



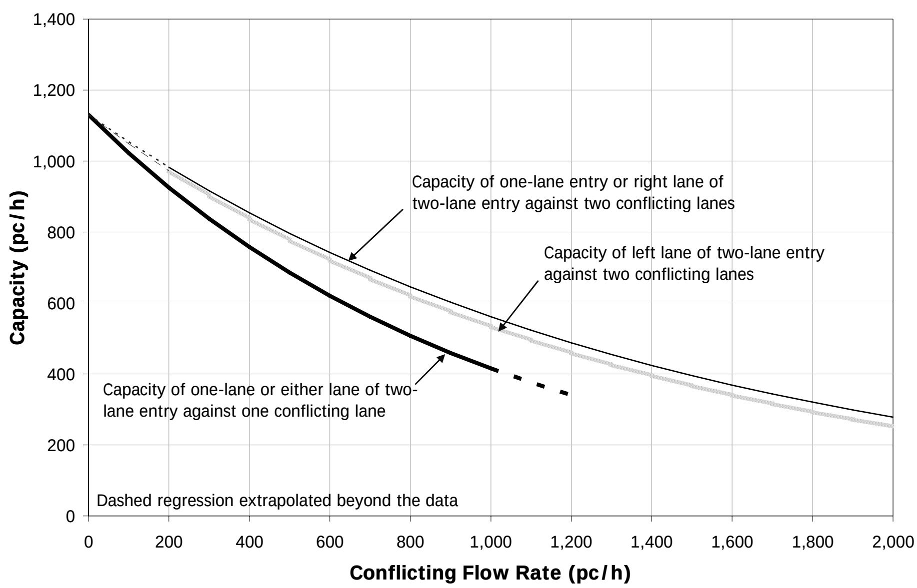
Field data (*1*) have found that drivers in the left lane have longer critical headways than drivers in the right lane. As a result, the capacity of the left lane is lower. Note that this research was able to observe sustained-queue conditions for only the right lane; Equation 21-4 represents a regression best fit that is also consistent with observed critical headways. The left-lane capacity given in Equation 21-5 is based on observed critical headways under both queued and nonqueued conditions.

The calculated capacities for each lane in passenger car equivalents per hour will be adjusted back to vehicles per hour, as described later in this section.

Exhibit 21-7 presents a plot showing Equation 21-1, Equation 21-4, and Equation 21-5. The dashed lines represent portions of the curves that lie outside the range of observed field data.

Exhibit 21-7
Capacity of Single-Lane and Multilane Entries

LIVE GRAPH
Click here to view



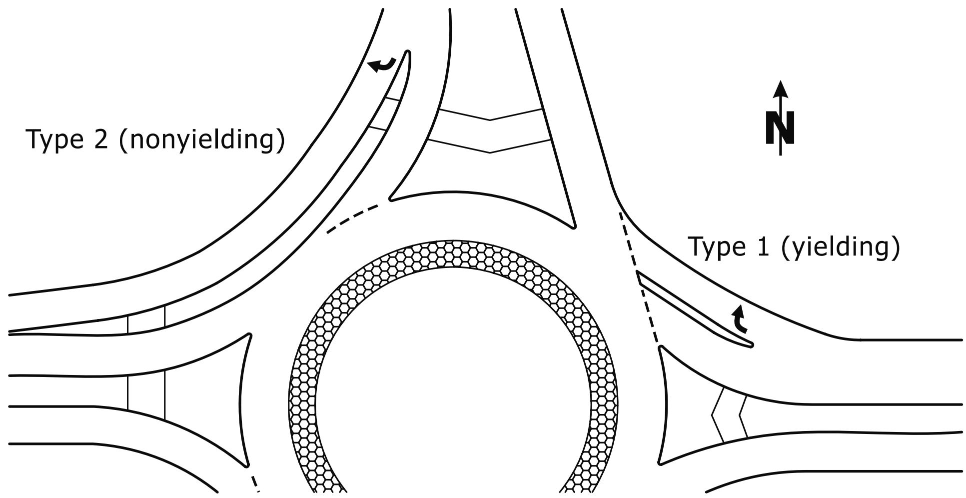
Right-Turn Bypass Lanes

Two common types of right-turn bypass lanes are used at both single-lane and multilane roundabouts. These are illustrated in Exhibit 21-8.

The bypass lane capacity procedure does not include the effect of drivers yielding to pedestrians.

The following sections describe each type of bypass lane. Note that in the United States, drivers in both types of bypass lane would generally be required to yield to pedestrians crossing the bypass lane. The capacity effect of drivers yielding to pedestrians has not been included in this analysis procedure.

Exhibit 21-8
Right-Turn Bypass Lanes



Type 1 (Yielding Bypass Lane)

A Type 1 bypass lane terminates at a high angle, with right-turning traffic yielding to exiting traffic. Right-turn bypass lanes were not explicitly included in the recent national research. However, the capacity of a yield bypass lane may be approximated by using one of the capacity formulas given previously by treating the exiting flow from the roundabout as the circulatory flow and treating the flow in the right-turn bypass lane as the entry flow.

The capacity for a bypass lane opposed by one exiting lane can be approximated by using Equation 21-6:

Equation 21-6

$$c_{\text{bypass},pce} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{ex,pce}}$$

The capacity for a bypass lane opposed by two exiting lanes can be approximated by using Equation 21-7:

Equation 21-7

$$c_{\text{bypass},pce} = 1{,}130e^{\left(-0.7\times10^{-3}\right)v_{ex,pce}}$$

where

$c_{\text{bypass},pce}$ = capacity of the bypass lane, adjusted for heavy vehicles (pc/h); and

$v_{ex,pce}$ = conflicting exiting flow rate (pc/h).

Type 2 (Nonyielding Bypass Lane)

A Type 2 bypass lane merges at a low angle with exiting traffic or forms a new lane adjacent to exiting traffic. The capacity of a merging bypass lane has not been assessed in the United States. Its capacity is expected to be relatively high due to a merging operation between two traffic streams at similar speeds.

Exit Capacity

German research (*6*) has suggested that the capacity of an exit lane, accounting for pedestrian and bicycle traffic in a typical urban area, is in the range of 1,200 to 1,300 vehicles per hour (veh/h). A Federal Highway Administration document used this information to provide guidance that exit flows exceeding 1,200 veh/h may indicate the need for a double-lane exit (*2*).

However, the analyst is cautioned to also evaluate exit lane requirements on the basis of vehicular lane numbers and arrangements. For example, a double-lane exit might be required to receive two through lanes in order to provide basic lane continuity along a corridor, regardless of the volume at the exit. Further guidance can be found elsewhere (2).

AUTOMOBILE MODE

The capacity of a given approach is computed by using the process illustrated in Exhibit 21-9.

Exhibit 21-9
Roundabout Analysis Methodology

Step 1: Convert movement demand volumes to flow rates

↓

Step 2: Adjust flow rates for heavy vehicles

↓

Step 3: Determine circulating and exiting flow rates

↓

Step 4: Determine entry flow rates by lane

↓

Step 5: Determine the capacity of each entry lane and bypass lane as appropriate in passenger car equivalents

↓

Step 6: Determine pedestrian impedance to vehicles

↓

Step 7: Convert lane flow rates and capacities into vehicles per hour

↓

Step 8: Compute the volume-to-capacity ratio for each lane

↓

Step 9: Compute the average control delay for each lane

↓

Step 10: Determine LOS for each lane on each approach

↓

Step 11: Compute the average control delay and determine LOS for each approach and the roundabout as a whole

↓

Step 12: Compute 95th percentile queues for each lane

Step 1: Convert Movement Demand Volumes to Flow Rates

For an analysis of existing conditions in which the peak 15-min period can be measured in the field, the volumes for the peak 15-min period are converted to a peak 15-min demand flow rate by multiplying the peak 15-min volumes by 4.

For analysis of projected conditions or when 15-min data are not available, hourly demand volumes for each movement are converted to peak 15-min demand flow rates in vehicles per hour, as shown in Equation 21-8, through the use of a peak hour factor for the intersection:

Equation 21-8

$$v_i = \frac{V_i}{PHF}$$

where

v_i = demand flow rate for movement *i* (veh/h),

V_i = demand volume for movement *i* (veh/h), and

PHF = peak hour factor.

Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicle stream characteristics by using factors given in Exhibit 21-10.

Exhibit 21-10
Passenger Car Equivalencies

Vehicle Type	Passenger Car Equivalent, E_T
Passenger car	1.0
Heavy vehicle	2.0

The calculation to incorporate these values is given in Equation 21-9 and Equation 21-10:

Equation 21-9

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Equation 21-10

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right)}$$

where

$v_{i,pce}$ = demand flow rate for movement *i* (pc/h),

v_i = demand flow rate for movement *i* (veh/h),

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of demand volume that consists of heavy vehicles, and

E_T = passenger car equivalent for heavy vehicles.

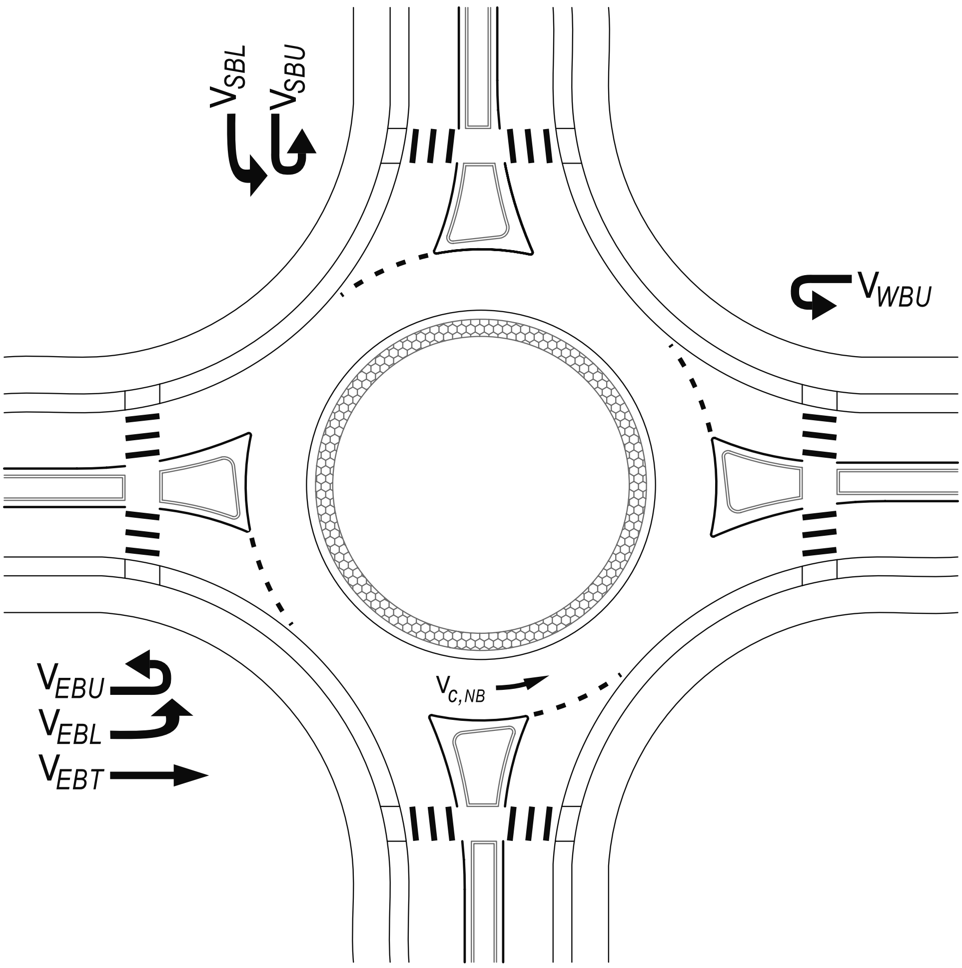
Step 3: Determine Circulating and Exiting Flow Rates

Circulating and exiting flow rates are calculated for each roundabout leg. Although the following sections present a numerical methodology for a four-leg roundabout, this methodology can be extended to any number of legs.

Circulating Flow Rate

The circulating flow opposing a given entry is defined as the flow conflicting with the entry flow (i.e., the flow passing in front of the splitter island next to the subject entry). The circulating flow rate calculation for the northbound circulating flow rate is illustrated in Exhibit 21-11 and numerically in Equation 21-11. All flows are in passenger car equivalents.

Exhibit 21-11
Calculation of Circulating Flow



Equation 21-11

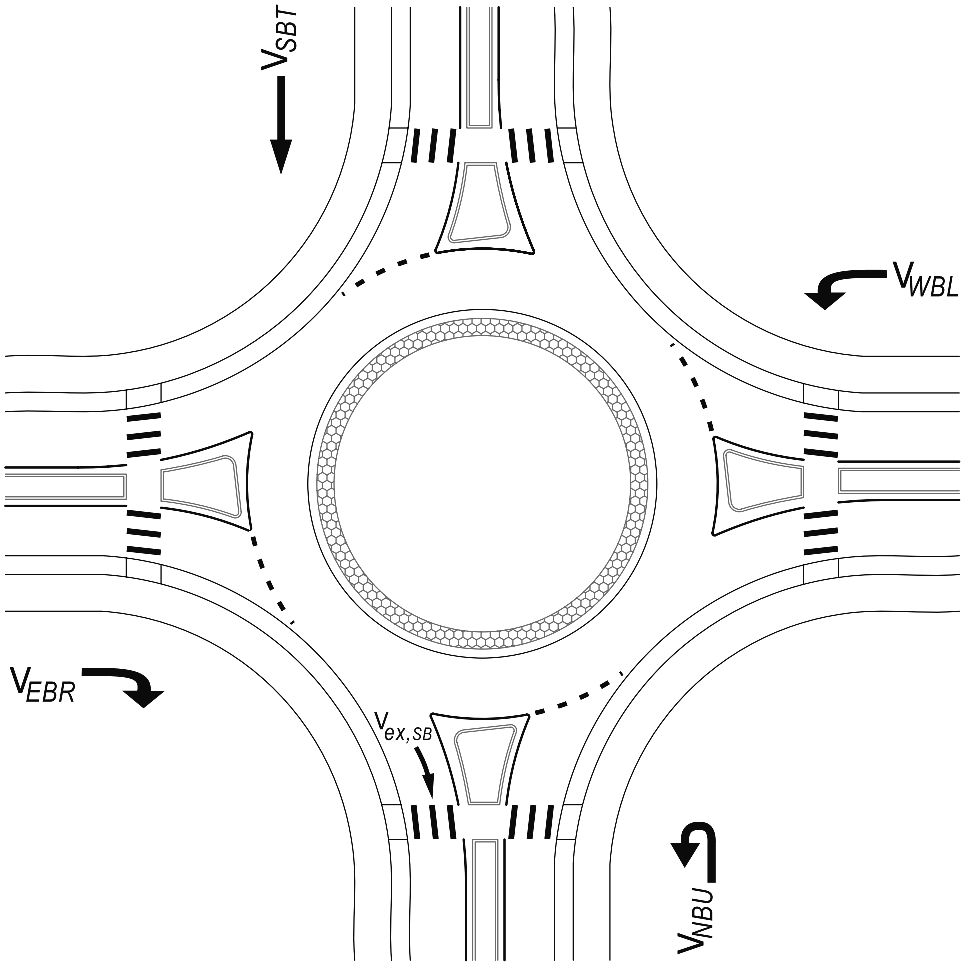
$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Exiting Flow Rate

The exiting flow rate for a given leg is used primarily in the calculation of conflicting flow for right-turn bypass lanes. The exiting flow calculation for the southbound exit is illustrated in Exhibit 21-12 and numerically in Equation 21-12. If a bypass lane is present on the immediate upstream entry, the right-turning flow using the bypass lane is deducted from the exiting flow. All flows are in passenger car equivalents.

If a bypass lane is present on the immediate upstream entry, the right-turning flow using the bypass lane is deducted from the exiting flow.

Exhibit 21-12
Calculation of Exiting Flow



Equation 21-12

$$v_{ex,pce} = v_{NBU,pce} + v_{WBL,pce} + v_{SBT,pce} + v_{EBR,pce} - v_{EBR,pce,bypass}$$

Step 4: Determine Entry Flow Rates by Lane

For single-lane entries, the entry flow rate is the sum of all movement flow rates using that entry. For multilane entries or entries with bypass lanes, or both, the following procedure may be used to assign flows to each lane:

1. If a right-turn bypass lane is provided, the flow using the bypass lane is removed from the calculation of the roundabout entry flows.
2. If only one lane is available for a given movement, the flow for that movement is assigned only to that lane.
3. The remaining flows are assumed to be distributed across all lanes, subject to the constraints imposed by any designated or de facto lane assignments and any observed or estimated lane utilization imbalances.

A de facto lane is one designated for multiple movements but that may operate as an exclusive lane due to a dominant movement demand. A common example is a left–through lane with a left-turn flow rate that greatly exceeds the through flow rate.

Five generalized multilane cases may be analyzed with this procedure. For cases in which a movement may use more than one lane, a check should first be made to determine what the assumed lane configuration may be. This may differ from the designated lane assignment based on the specific turning movement patterns being analyzed. These assumed lane assignments are given in Exhibit

21-13. For intersections with a different number of legs, the analyst should exercise reasonable judgment in assigning volumes to each lane.

Exhibit 21-13
Assumed (de facto) Lane Assignments

Designated Lane Assignment	Assumed Lane Assignment
LT, TR	If $v_U + v_L > v_T + v_{R,e}$: L, TR (de facto left-turn lane) If $v_{R,e} > v_U + v_L + v_T$: LT, R (de facto right-turn lane) Else LT, TR
L, LTR	If $v_T + v_{R,e} > v_U + v_L$: L, TR (de facto through–right lane) Else L, LTR
LTR, R	If $v_U + v_L + v_T > v_{R,e}$: LT, R (de facto left–through lane) Else LTR, R

Notes: v_U, v_L, v_T, and $v_{R,e}$ are the U-turn, left-turn, through, and nonbypass right-turn flow rates using a given entry, respectively.
L = left, LT = left–through, TR = through–right, LTR = left–through–right, and R = right.

On the basis of the assumed lane assignment for the entry and the lane utilization effect described above, flow rates can be assigned to each lane by using the formulas given in Exhibit 21-14. In this exhibit, $\%RL$ is the percentage of entry traffic using the right lane, $\%LL$ is the percentage of entry traffic using the left lane, and $\%LL + \%RL = 1$.

Exhibit 21-14
Volume Assignments for Two-Lane Entries

Case	Assumed Lane Assignment	Left Lane	Right Lane
1	L, TR	$v_U + v_L$	$v_T + v_{R,e}$
2	LT, R	$v_U + v_L + v_T$	$v_{R,e}$
3	LT, TR	$(\%LL)v_e$	$(\%RL)v_e$
4	L, LTR	$(\%LL)v_e$	$(\%RL)v_e$
5	LTR, R	$(\%LL)v_e$	$(\%RL)v_e$

Notes: v_U, v_L, v_T, and $v_{R,e}$ are the U-turn, left-turn, through, and nonbypass right-turn flow rates using a given entry, respectively.
L = left, LT = left–through, TR = through–right, LTR = left–through–right, and R = right.

Further discussion of lane use at multilane roundabouts, including conditions that may create unequal lane use, can be found in Chapter 33, Roundabouts: Supplemental, located in HCM Volume 4.

Step 5: Determine the Capacity of Each Entry Lane and Bypass Lane as Appropriate in Passenger Car Equivalents

The capacity of each entry lane and bypass lane is calculated by using the capacity equations discussed previously. Capacity equations for entry lanes are summarized in Exhibit 21-15; capacity equations for bypass lanes are summarized in Exhibit 21-16.

Exhibit 21-15
Capacity Equations for Entry Lanes

Entering Lanes	Conflicting Circulating Lanes	Capacity Equation
1	1	Equation 21-1
2	1	Each lane: Equation 21-2
1	2	Equation 21-3
2	2	Right lane: Equation 21-4; left lane: Equation 21-5

Exhibit 21-16
Capacity Equations for Bypass Lanes

Conflicting Exiting Lanes	Capacity Equation
1	Equation 21-6
2	Equation 21-7

Step 6: Determine Pedestrian Impedance to Vehicles

Pedestrian traffic can reduce the vehicular capacity of a roundabout entry if sufficient pedestrians are present and they assert the right-of-way typically granted pedestrians in most jurisdictions. Under high vehicular conflicting flows, pedestrians typically pass between queued vehicles on entry and thus have negligible additional impact on vehicular entry capacity. However, under low vehicular conflicting flows, pedestrians can effectively function as additional conflicting vehicles and thus reduce the vehicular capacity of the entry. The effect of pedestrians is more pronounced with increased pedestrian volume.

For one-lane roundabout entries, the model shown in Exhibit 21-17 can be used to approximate this effect (*6*). These equations are illustrated in Exhibit 21-18 and are based on the assumption that pedestrians have absolute priority.

Exhibit 21-17
Model of Entry Capacity Adjustment Factor for Pedestrians Crossing a One-Lane Entry (Assuming Pedestrian Priority)

Case	One-Lane Entry Capacity Adjustment Factor for Pedestrians
If $v_{c,pce} > 881$	$f_{ped} = 1$
Else if $n_{ped} \leq 101$	$f_{ped} = 1 - 0.000137 n_{ped}$
Else	$f_{ped} = \dfrac{1{,}119.5 - 0.715 v_{c,pce} - 0.644 n_{ped} + 0.00073 v_{c,pce} n_{ped}}{1{,}068.6 - 0.654 v_{c,pce}}$

where

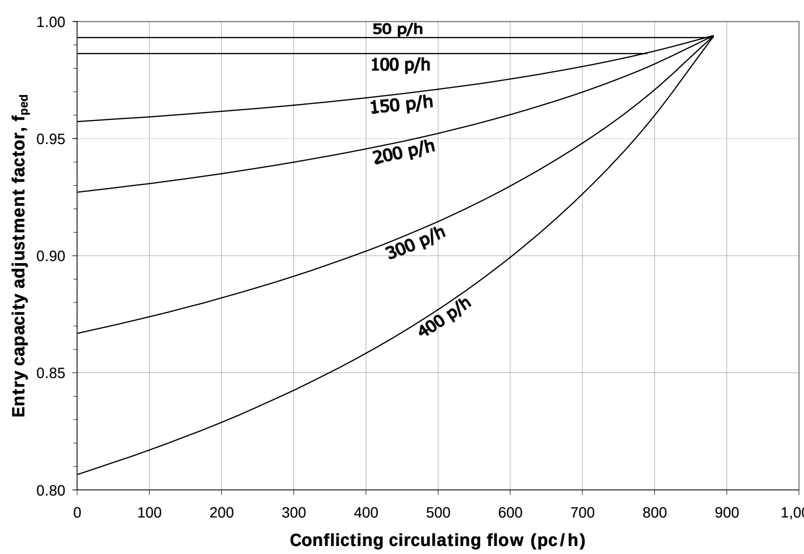
f_{ped} = entry capacity adjustment factor for pedestrians,

n_{ped} = number of conflicting pedestrians per hour (p/h), and

$v_{c,pce}$ = conflicting vehicular flow rate in the circulatory roadway, pc/h.

Exhibit 21-18
Illustration of Entry Capacity Adjustment Factor for Pedestrians Crossing a One-Lane Entry (Assuming Pedestrian Priority)

LIVE GRAPH
Click here to view



For two-lane entries, the model shown in Exhibit 21-19 can be used to approximate this effect (*6*). These equations are illustrated in Exhibit 21-20 and share the assumption as before that pedestrians have absolute priority.

Exhibit 21-19
Model of Entry Capacity Adjustment Factor for Pedestrians Crossing a Two-Lane Entry (Assuming Pedestrian Priority)

Case	Two-Lane Entry Capacity Adjustment Factor for Pedestrians
If $n_{ped} < 100$	$f_{ped} = \min\left[1 - \frac{n_{ped}}{100}\left(1 - \frac{1{,}260.6 - 0.329 v_{c,pce} - 0.381 \times 100}{1{,}380 - 0.5 v_{c,pce}}\right), 1\right]$
Else	$f_{ped} = \min\left[\frac{1{,}260.6 - 0.329 v_{c,pce} - 0.381 n_{ped}}{1{,}380 - 0.5 v_{c,pce}}, 1\right]$

where

f_{ped} = entry capacity adjustment factor for pedestrians,

n_{ped} = number of conflicting pedestrians (p/h), and

$v_{c,pce}$ = conflicting vehicular flow rate in the circulatory roadway (pc/h).

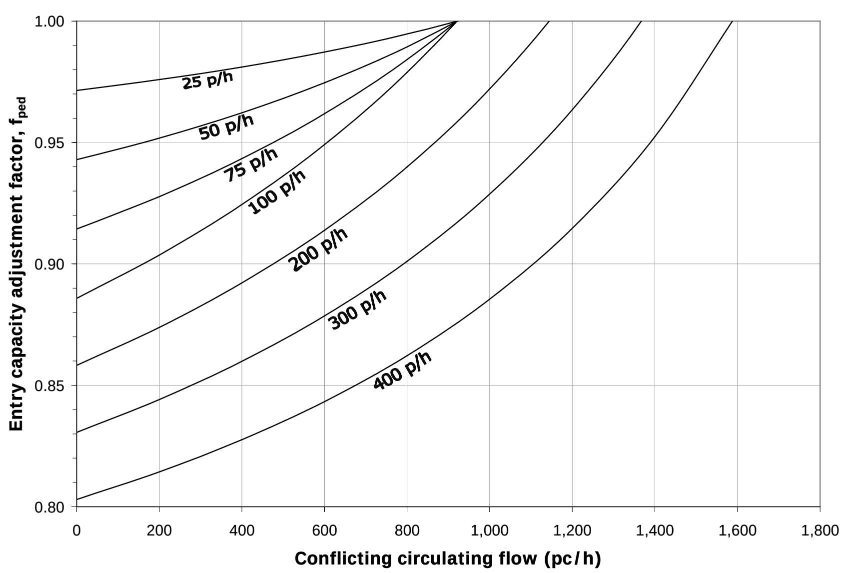


Exhibit 21-20
Illustration of Entry Capacity Adjustment Factor for Pedestrians Crossing a Two-Lane Entry (Assuming Pedestrian Priority)

LIVE GRAPH
Click here to view

Step 7: Convert Lane Flow Rates and Capacities into Vehicles per Hour

The flow rate for a given lane is converted back to vehicles per hour by multiplying the passenger-car-equivalent flow rate computed in the previous step by the heavy-vehicle factor for the lane as shown in Equation 21-13:

Equation 21-13

$$v_i = v_{i,PCE} f_{HV,e}$$

where

v_i = flow rate for lane *i* (veh/h),

$v_{i,PCE}$ = flow rate for lane *i* (pc/h), and

$f_{HV,e}$ = heavy-vehicle adjustment factor for the lane (see below).

Similarly, the capacity for a given lane is converted back to vehicles per hour as shown in Equation 21-14:

Equation 21-14

$$c_i = c_{i,PCE} f_{HV,e} f_{ped}$$

where

c_i = capacity for lane i (veh/h),

$c_{i,PCE}$ = capacity for lane i (pc/h),

$f_{HV,e}$ = heavy-vehicle adjustment factor for the lane (see below), and

f_{ped} = pedestrian impedance factor.

The heavy-vehicle adjustment factor for each entry lane can be approximated by taking a weighted average of the heavy-vehicle adjustment factors for each movement entering the roundabout (excluding a bypass lane if present) weighted by flow rate, as shown in Equation 21-15:

Equation 21-15

$$f_{HV,e} = \frac{f_{HV,U} v_{U,PCE} + f_{HV,L} v_{L,PCE} + f_{HV,T} v_{T,PCE} + f_{HV,R,e} v_{R,e,PCE}}{v_{U,PCE} + v_{L,PCE} + v_{T,PCE} + v_{R,e,PCE}}$$

where

$f_{HV,e}$ = heavy-vehicle adjustment factor for the entry lane,

$f_{HV,i}$ = heavy-vehicle adjustment factor for movement i, and

$v_{i,PCE}$ = demand flow rate for movement i (pc/h).

If specific lane-use assignment by heavy vehicles is known, heavy-vehicle adjustment factors can be calculated separately for each lane.

Pedestrian impedance is discussed later in this chapter.

Step 8: Compute the Volume-to-Capacity Ratio for Each Lane

For a given lane, the volume-to-capacity ratio x is calculated by dividing the lane's calculated capacity into its demand flow rate, as shown in Equation 21-16. Both input values are in vehicles per hour.

Equation 21-16

$$x_i = \frac{v_i}{c_i}$$

where

x_i = volume-to-capacity ratio of the subject lane i,

v_i = demand flow rate of the subject lane i (veh/h), and

c_i = capacity of the subject lane i (veh/h).

Step 9: Compute the Average Control Delay for Each Lane

Delay data collected for roundabouts in the United States suggest that control delays can be predicted in a manner generally similar to that used for other unsignalized intersections. Equation 21-17 shows the model that should be used to estimate average control delay for each lane of a roundabout approach:

$$d = \frac{3{,}600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{450T}}\,\right] + 5 \times \min[x,1]$$

Equation 21-17

The third term of this equation uses the calculated volume-to-capacity ratio or 1, whichever is less.

where

d = average control delay (s/veh),

x = volume-to-capacity ratio of the subject lane,

c = capacity of the subject lane (veh/h), and

T = time period (h) (T = 0.25 h for a 15-min analysis).

Equation 21-17 is the same as that for STOP-controlled intersections except that the "+ 5" term has been modified. This modification is necessary to account for the YIELD control on the subject entry, which does not require drivers to come to a complete stop when there is no conflicting traffic. At higher volume-to-capacity ratios, the likelihood of coming to a complete stop increases, thus causing behavior to resemble STOP control more closely.

Average control delay for a given lane is a function of the lane's capacity and degree of saturation. The analytical model used above to estimate average control delay assumes that there is no residual queue at the start of the analysis period. If the degree of saturation is greater than about 0.9, average control delay is significantly affected by the length of the analysis period. In most cases, the recommended analysis period is 15 min. If demand exceeds capacity during a 15-min period, the delay results calculated by the procedure may not be accurate due to the likely presence of a queue at the start of the time period. In addition, the conflicting demand for movements downstream of the movement operating over capacity may not be fully realized (in other words, the flow cannot get past the oversaturated entry and thus cannot conflict with a downstream entry). In these cases, an iterative approach that accounts for this effect and the carryover of queues from one time period to the next may be considered, as discussed elsewhere (*7*).

Step 10: Determine LOS for Each Lane on Each Approach

The LOS for each lane on each approach is determined by using Exhibit 21-1 and the computed or measured values of control delay.

Step 11: Compute the Average Control Delay and Determine LOS for Each Approach and the Roundabout as a Whole

The control delay for an approach is calculated by computing a weighted average of the delay for each lane on the approach, weighted by the volume in each lane. The calculation is shown in Equation 21-18. Note that the volume in the bypass lane should be included in the delay calculation for the approach. The LOS for each approach is determined by using Exhibit 21-1 and the computed or measured values of control delay.

Equation 21-18

$$d_{\text{approach}} = \frac{d_{LL}v_{LL} + d_{RL}v_{RL} + d_{\text{bypass}}v_{\text{bypass}}}{v_{LL} + v_{RL} + v_{\text{bypass}}}$$

The control delay for the intersection as a whole is similarly calculated by computing a weighted average of the delay for each approach, weighted by the volume on each approach. This is shown in Equation 21-19. The LOS for the intersection is determined by using Exhibit 21-1 and the computed or measured values of control delay.

Equation 21-19

$$d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

$d_{\text{intersection}}$ = control delay for the entire intersection (s/veh),

d_i = control delay for approach i (s/veh), and

v_i = flow rate for approach i (veh/h).

Step 12: Compute 95th Percentile Queues for Each Lane

The 95th percentile queue for a given lane on an approach is calculated by using Equation 21-20:

Equation 21-20

$$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{150T}}\right]\left(\frac{c}{3{,}600}\right)$$

where

Q_{95} = 95th percentile queue (veh),

x = volume-to-capacity ratio of the subject lane,

c = capacity of the subject lane (veh/h), and

T = time period (h) ($T = 1$ for a 1-h analysis, $T = 0.25$ for a 15-min analysis).

The queue length calculated for each lane should be checked against the available storage. The queue in each lane may interact with adjacent lanes in one or more ways:

- If queues in adjacent lanes exceed the available storage, the queue in the subject lane may be longer than anticipated due to additional queuing from the adjacent lane.
- If queues in the subject lane exceed the available storage for adjacent lanes, the adjacent lane may be starved by the queue in the subject lane.

Should one or more of these conditions occur, a sensitivity analysis can be conducted with the methodology by varying the demand in each lane. The analyst may also use an alternative tool that is sensitive to lane-by-lane effects, as discussed in this chapter's Applications section.

PEDESTRIAN MODE

Limited research has been performed to date in the United States on the operational impacts of vehicular traffic on pedestrians at roundabouts. In the United States, pedestrians have the right-of-way either after entering a crosswalk or as they are about to enter the crosswalk, depending on specific state law. This is somewhat different from other countries that may establish absolute pedestrian right-of-way in some situations (typically urban) and absolute vehicular right-of-way in others (typically rural).

Much of the recent research focus on pedestrians in the United States has been in the area of assessing accessibility for pedestrians with vision disabilities. Research has found that some roundabouts present a challenge for blind and visually impaired pedestrians relative to sighted pedestrians, thus potentially bringing them out of compliance with the Americans with Disabilities Act (*8*). A variety of treatments has been or is being considered to improve roundabouts' accessibility to this group of pedestrians, including various types of signalization of pedestrian crossings. The analysis of these treatments can in some cases be performed by simple analytical methods presented in the HCM (e.g., the analysis procedure for the pedestrian mode in Chapter 19). However, in many cases, alternative tools will produce more accurate results. These are discussed later in this chapter.

Techniques to analyze the operational performance of pedestrians as provided in Chapter 19, Two-Way STOP-Controlled Intersections, can be applied with care at roundabouts. As noted in that chapter, vehicular yielding rates vary depending on crossing treatment, number of lanes, posted speed limit, and within individual sites (*9*). This makes modeling of pedestrian interactions imprecise. As a result, models to analyze vehicular effects on pedestrian travel should be applied with caution.

BICYCLE MODE

Use a passenger-car-equivalent factor of 0.5 for bicycles when treating them as motor vehicles.

As of the publication date of this edition of the HCM, no methodology specific to bicyclists has been developed to assess the performance of bicyclists at roundabouts, as few data are available in the United States to support model calibration. Depending on individual comfort level, ability, geometric conditions, and traffic conditions, bicyclists may either circulate as a motor vehicle or as a pedestrian. If bicyclists are circulating as motor vehicles, their effect can be approximated by combining bicyclist flow rates with other vehicles by using a passenger-car-equivalent factor of 0.5 (*2*). If bicyclists are circulating as pedestrians, their effect can be analyzed by using the methodology described previously for pedestrians. Further guidance on accommodating bicyclists at roundabouts can be found elsewhere (*2*).

3. APPLICATIONS

DEFAULT VALUES

No default values have been developed specifically for roundabouts. However, a comprehensive presentation of potential default values for interrupted-flow facilities is available (*10*), with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of peak hour factor and percent heavy vehicles. Recommendations are based on geographical region, population, and time of day. All general default values for interrupted-flow facilities may be applied to the analysis of roundabouts in the absence of field data or projections of conditions.

Demand volumes as well as the number and configuration of lanes at a roundabout are site-specific and thus do not lend themselves to default values. The following default values may be applied to a roundabout analysis:

- Peak hour factor = 0.92, and
- Percent heavy vehicles = 3%.

Default values for lane utilization on two-lane roundabout approaches are not provided in the above reference (*10*). In these cases, in the absence of field data, the effect of lane utilization imbalance can be approximated by using the assumed values given in Exhibit 21-21.

Exhibit 21-21
Assumed Default Values for Lane Utilization for Two-Lane Approaches

Lane Configuration	% Traffic in Left Lane[a]	% Traffic in Right Lane[a]
Left–through + through–right	0.47	0.53
Left–through–right + right	0.47	0.53
Left + left–through–right	0.53	0.47

Notes: [a] These values are generally consistent with observed values for through movements at signalized intersections. These values should be applied with care, particularly under conditions estimated to be near capacity.

Obviously, as the number of default values used in any analysis increases, the analysis result becomes more approximate and may be significantly different from the actual outcome, depending on local conditions.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary engineering analysis.

Operational Analysis

Operational analysis takes traffic flow data and geometric configurations as input to determine operational performance.

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement (in vehicles per hour), heavy-vehicle percentages for each approach, peak hour factor for all hourly demand volumes (if not provided as 15-min volumes), and lane configuration. The outputs of an operational analysis will be estimates of capacity and control delay. The steps of the methodology,

described in the Methodology section, are followed directly without modification.

Design Analysis

The operational analysis methodology described earlier in this chapter can be used for design purposes by using a given set of traffic flow data to determine iteratively the number and configuration of lanes that would be required to produce a given LOS.

Design analysis is used to determine the geometric configuration of a roundabout to produce a desired operational performance.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of a roundabout. To estimate LOS for a future time horizon, a planning analysis based on the operational method is used. The planning method uses all the geometric and traffic flow data required for an operational analysis, and the computations are identical. However, input variables for percent heavy vehicles and peak hour factor are typically estimated (or defaults used) when planning applications are performed.

Planning and preliminary engineering analysis is used to evaluate future conditions for which assumptions and estimates must be made.

CALIBRATION OF CAPACITY MODEL

The capacity models presented previously can be generalized by using the expressions in Equation 21-21 through Equation 21-23 as follows:

Equation 21-21

$$c_{pce} = Ae^{(-B\,v_c)}$$

Equation 21-22

$$A = \frac{3{,}600}{t_f}$$

Equation 21-23

$$B = \frac{t_c - (t_f/2)}{3{,}600}$$

where

c_{pce} = lane capacity, adjusted for heavy vehicles (pc/h),

v_c = conflicting flow (pc/h),

t_c = critical headway (s), and

t_f = follow-up headway (s).

Field measures of critical headway and follow-up headway can be used to calibrate the capacity models.

Therefore, the capacity model can be calibrated by using two parameters: the critical headway t_c and the follow-up headway t_f. An example illustrating this procedure is provided in Chapter 33, Roundabouts: Supplemental.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for the application of alternative tools to the analysis of roundabouts. The reader should also be familiar with the information and guidance on the design and evaluation of roundabouts (*2*, *3*).

Two modeling approaches are used in the types of alternative tools commonly applied:

- *Deterministic intersection models.* These models represent vehicle flows as flow rates and are sensitive to various flow and geometric features of the roundabout, including lane numbers and arrangements or specific geometric dimensions (e.g., entry width, inscribed circle diameter), or both. The majority of these models are anchored to research conducted outside the United States (e.g., *11–14*). Some software implementations may include more than one model or employ extensions beyond the original fundamental research conducted within a particular country. Some deterministic models can model an entire network of intersections but generally assume no interaction effects between intersections, thus potentially limiting their application.
- *Stochastic network models.* These models represent vehicle flows by simulating individual vehicles and their car-following, lane-choice, and gap-acceptance decisions. The models are based on a variety of fundamental research studies on driver behavior (e.g., *15*, *16*). By their nature, most stochastic models used for roundabouts can model an entire network of intersections, thus making them capable of modeling a broader range of problems. However, their data requirements are typically more intensive than for the deterministic intersection models. Most stochastic models are implemented in microsimulation tools.

Strengths of the HCM Procedure

The procedures in this chapter were based on extensive research supported by a significant quantity of field data. They have evolved over several years and represent a body of expert consensus. They produce unique deterministic results for a given set of inputs, and the capacity of each approach is an explicit part of the results. Alternative tools based on deterministic intersection models also produce a unique set of results, including capacities, for a given set of inputs, while those based on simulation may produce different results based on different random number sequences. Unique results from an analysis tool are important for some purposes such as development impact review.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

The procedures presented in this chapter cover many of the typical situations that a user may encounter in practice. However, there are sometimes applications for which alternative tools can produce a more accurate analysis. The following limitations, stated earlier in this chapter, may be addressed by using available simulation tools. The conditions beyond the scope of this chapter that are treated explicitly by alternative tools include

- Adjacent signals or roundabouts,
- Priority reversal under extremely high flows,
- High pedestrian or bicycle activity levels,
- More than two entry lanes on an approach, or

- Flared entry lanes.

A few of the more common applications of alternative tools to overcome the limitations of the procedures presented in this chapter will now be discussed.

Interaction Effects with Other Traffic Control Devices

Several common situations can be modeled with alternative tools:

- *Pedestrian signals or hybrid beacons at roundabout crosswalks.* These devices, described in detail in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (*17*), can be used in a variety of applications, including the following:
    - High vehicle flows in which naturally occurring gaps in vehicle traffic or vehicular yielding for pedestrians is insufficient;
    - High pedestrian flows in which unrestricted pedestrian crossing activity may create insufficient capacity for motor vehicles; and
    - Crossing situations in which pedestrians with vision or other impairments may not receive equivalent access to the crossing. This is a legal requirement in the United States under the Americans with Disabilities Act and is regulated by the U.S. Access Board (*8*).
- *Metering signals on roundabout approaches.* These signals are sometimes used in applications in which a dominant entering flow reduces downstream entry capacity to zero or nearly zero. A metering signal can create gaps in the dominant flow at regular intervals or as dictated by queuing at the downstream entry.
- *Signals used to give priority to other users.* These applications include at-grade rail crossings, emergency vehicle signals, and others.
- *Nearby intersections or traffic control devices at which queues or lane use effects interact.* These nearby intersections can have any type of control, including signalization, STOP control, or YIELD control (as at another roundabout). Applications could also include nonintersection treatments such as freeway ramp meters.

While some deterministic intersection tools can model these situations, they are often treated more satisfactorily by using stochastic network models.

Flared Entries or Short-Lane Applications

Flared entries or short-lane applications are sometimes used at roundabouts to add capacity at the entry without substantially widening the approach upstream of the entry. Common applications include flaring from one lane to two lanes at the entry or from two lanes to three lanes, although some international research has found capacity sensitivity to flaring in sub-lane-width increments (*13*).

The methodology presented in this chapter provides a mechanism for flagging conditions under which queues for a given lane may exceed available storage or block access to adjacent lanes. Alternative tools may provide more accurate modeling of these situations.

Three-Lane Roundabouts

Three-lane roundabouts are not included in the methodology described in this chapter but can be analyzed by a number of alternative tools. Note that no data for three-lane roundabouts are available in the source material (*1*) for this chapter's methodology, so the analyst should use care in estimating calibration parameters.

Adjustment of Simulation Parameters to the HCM Results

Calibration of any model used to analyze roundabouts is essential in producing realistic results that are consistent with field data. Ideally, field data should be used for calibration. For situations involving the assessment of hypothetical or proposed alternatives for which no field data exist, alternative tool results may be made more compatible with HCM results by adjusting alternative tool parameters to obtain a better match with the results obtained from the HCM procedures as follows:

- *Deterministic intersection models.* Typical calibration parameters for deterministic models include global adjustment factors that shift or shape the capacity model used by the model. These include adjustments to the intercept and slope of linear models or other shaping parameters of more complex analytical forms.
- *Stochastic network models.* Calibration of stochastic models is more challenging than for deterministic models because some calibration factors, such as factors related to driver aggressiveness, often apply globally to all elements of the network and not just to roundabouts. In other cases, the specific coding of the model can be fine-tuned to reflect localized driver behavior, including look-ahead points for gap acceptance and locations for discretionary and mandatory lane changes.

Step-by-Step Recommendations for Applying Alternative Tools

The following steps should be taken in applying an alternative tool in the analysis of roundabouts:

1. Identify the limitations of the HCM procedures that dictate the use of alternative tools.
2. Decide between a microscopic and a macroscopic modeling approach.
3. If possible, develop a simpler configuration that can be analyzed by the HCM procedures. Analyze the simple configuration by using both the HCM and the selected alternative tool. Make adjustments to the alternative tool parameters to obtain a better match with the HCM results.
4. Perform the analysis of the full configuration by using the alternative tool.
5. Interpret and present the results.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 29, Urban Street Facilities: Supplemental, includes an example of the application of a simulation tool to assess the effect of using a roundabout within a coordinated arterial signal system. The interactions between the roundabout and the arterial system are examined by using signal timing plans with different progression characteristics.

4. EXAMPLE PROBLEMS

Exhibit 21-22
List of Example Problems

Example Problem	Description	Application
1	Single-lane roundabout with bypass lanes	Operational analysis
2	Multilane roundabout	Operational analysis

EXAMPLE PROBLEM 1: SINGLE-LANE ROUNDABOUT WITH BYPASS LANES

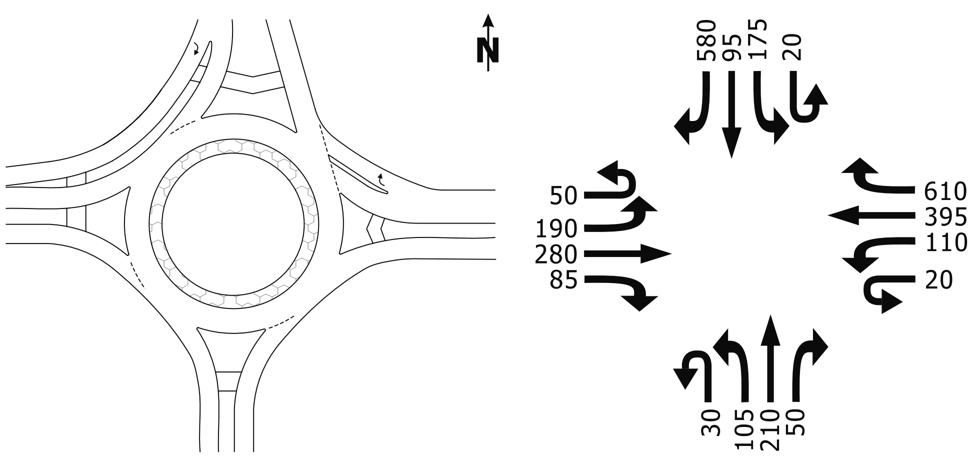
This is an example of an operational analysis. It uses traffic data and geometric characteristics to determine capacities, control delay, and LOS.

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Four legs,
- One-lane entries on each leg,
- A westbound right-turn bypass lane that yields to exiting vehicles,
- A southbound right-turn bypass lane that forms its own lane adjacent to exiting vehicles,
- Percent heavy vehicles for all movements = 2%,
- Peak hour factor = 0.94,
- Demand volumes and lane configurations as shown in Exhibit 21-23, and
- 50 p/h across the south leg and negligible pedestrian activity across the other three legs.

Exhibit 21-23
Demand Volumes and Lane Configurations for Example Problem 1



Comments

All input parameters are known, so no default values are needed or used.

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement volume given in the problem is converted to a demand flow rate by dividing by the peak hour factor. As an example, the northbound left-turn volume is converted to a flow rate as follows:

$$v_{NBL} = \frac{V_{NBL}}{PHF} = \frac{105}{0.94} = 112 \text{ pc/h}$$

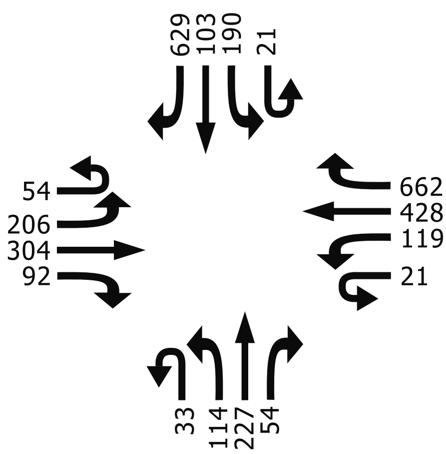
Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicle stream characteristics as follows (northbound left turn illustrated):

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.02(2-1)} = 0.980$$

$$v_{NBL,pce} = \frac{v_{NBL}}{f_{HV}} = \frac{112}{0.980} = 114 \text{ pc/h}$$

The resulting adjusted flow rates for all movements accounting for Steps 1 and 2 are therefore computed as follows:



Step 3: Determine Circulating and Exiting Flow Rates

The circulating and exiting flows are calculated for each leg. For the south leg (northbound entry), the circulating flow is calculated as follows:

$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$
$$v_{c,NB,pce} = 21 + 190 + 21 + 304 + 206 + 54 = 796 \text{ pc/h}$$

Similarly, $v_{c,SB,pce}$ = 769 pc/h, $v_{c,EB,pce}$ = 487 pc/h, and $v_{c,WB,pce}$ = 655 pc/h.

For this problem, one exit flow rate is needed: the northbound exit flow rate, which serves as the conflicting flow for the westbound bypass lane. Because all westbound right turns are assumed to use the bypass lane, they are excluded from the conflicting exit flow as follows:

$$v_{ex,pce,NB} = v_{SBU,pce} + v_{EBL,pce} + v_{NBT,pce} + v_{WBR,e,pce}$$
$$v_{ex,pce,NB} = 21 + 206 + 227 + 0 = 454 \text{ pc/h}$$

Step 4: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing the movement flow rates that enter the roundabout (without using a bypass lane). Because this is a single-lane roundabout, no lane-use calculations are needed.

The entry flow rates are calculated as follows, assuming that all right-turn volumes on the westbound and southbound approaches use the bypass lane provided and not the entry:

$$v_{e,NB,pce} = v_{NBU,pce} + v_{NBL,pce} + v_{NBT,pce} + v_{NBR,e,pce} = 33 + 114 + 227 + 54 = 428 \text{ pc/h}$$

$$v_{e,SB,pce} = v_{SBU,pce} + v_{SBL,pce} + v_{SBT,pce} + v_{SBR,e,pce} = 21 + 190 + 103 + 0 = 314 \text{ pc/h}$$

$$v_{e,EB,pce} = v_{EBU,pce} + v_{EBL,pce} + v_{EBT,pce} + v_{EBR,e,pce} = 54 + 206 + 304 + 92 = 656 \text{ pc/h}$$

$$v_{e,WB,pce} = v_{WBU,pce} + v_{WBL,pce} + v_{WBT,pce} + v_{WBR,e,pce} = 21 + 119 + 428 + 0 = 568 \text{ pc/h}$$

Step 5: Determine the Capacity of Each Entry Lane and Bypass Lane as Appropriate in Passenger Car Equivalents

By using the single-lane capacity equation (Equation 21-1), the capacity for each entry lane is given as follows:

$$c_{pce,NB} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce,NB}} = 1{,}130e^{(-1.0\times10^{-3})(796)} = 510 \text{ pc/h}$$

$$c_{pce,SB} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce,SB}} = 1{,}130e^{(-1.0\times10^{-3})(769)} = 524 \text{ pc/h}$$

$$c_{pce,EB} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce,EB}} = 1{,}130e^{(-1.0\times10^{-3})(487)} = 694 \text{ pc/h}$$

$$c_{pce,WB} = 1{,}130e^{(-1.0\times10^{-3})v_{c,pce,WB}} = 1{,}130e^{(-1.0\times10^{-3})(655)} = 587 \text{ pc/h}$$

By using the equation for a bypass lane opposed by a single exit lane (Equation 21-6), the capacity for the westbound bypass lane is given as follows:

$$c_{bypass,pce,WB} = 1{,}130e^{(-1.0\times10^{-3})v_{ex,pce,NB}} = 1{,}130e^{(-1.0\times10^{-3})(454)} = 718 \text{ pc/h}$$

Step 6: Determine Pedestrian Impedance to Vehicles

The south leg (northbound entry) has a conflicting pedestrian flow rate, n_{ped}, of 50 p/h. Therefore, the pedestrian impedance factor is calculated by using Exhibit 21-17 as follows:

$$f_{ped} = 1 - 0.000137n_{ped} = 1 - 0.000137(50) = 0.993$$

The other legs have negligible pedestrian activity and therefore have $f_{ped} = 1$.

Step 7: Convert Lane Flow Rates and Capacities into Vehicles per Hour

The capacity for a given lane is converted back to vehicles by first determining the heavy-vehicle adjustment factor for the lane and then multiplying it by the capacity in passenger car equivalents. For this example, since all turning movements on each entry have the same f_{HV}, each entry will also have the same f_{HV}, 0.980.

$$c_{NB} = c_{pce,NB} f_{HV,e,NB} f_{ped} = (510)(0.980)(0.993) = 497 \text{ veh/h}$$

$$c_{SB} = c_{pce,SB} f_{HV,e,SB} f_{ped} = (524)(0.980)(1) = 514 \text{ veh/h}$$

$$c_{EB} = c_{pce,EB} f_{HV,e,EB} f_{ped} = (694)(0.980)(1) = 680 \text{ veh/h}$$

$$c_{WB} = c_{pce,WB} f_{HV,e,WB} f_{ped} = (587)(0.980)(1) = 575 \text{ veh/h}$$

$$c_{\text{bypass},WB} = c_{\text{bypass},pce,NB} f_{HV,e,WB} f_{ped} = (718)(0.980)(1) = 704 \text{ veh/h}$$

Calculations for the entry flow rates are as follows:

$$v_{NB} = v_{pce,NB} f_{HV,e,NB} = (428)(0.980) = 420 \text{ veh/h}$$

$$v_{SB} = v_{pce,SB} f_{HV,e,SB} = (314)(0.980) = 308 \text{ veh/h}$$

$$v_{EB} = v_{pce,EB} f_{HV,e,EB} = (656)(0.980) = 643 \text{ veh/h}$$

$$v_{WB} = v_{pce,WB} f_{HV,e,WB} = (568)(0.980) = 557 \text{ veh/h}$$

$$v_{\text{bypass},WB} = v_{\text{bypass},pce,NB} f_{HV,e,WB} f_{ped} = (662)(0.980) = 649 \text{ veh/h}$$

Step 8: Compute the Volume-to-Capacity Ratio for Each Lane

The volume-to-capacity ratios for each entry lane are calculated as follows:

$$x_{NB} = \frac{420}{497} = 0.85$$

$$x_{SB} = \frac{308}{514} = 0.60$$

$$x_{EB} = \frac{643}{680} = 0.95$$

$$x_{WB} = \frac{557}{575} = 0.97$$

$$x_{bypass,WB} = \frac{649}{704} = 0.92$$

Step 9: Compute the Average Control Delay for Each Lane

The control delay for the northbound entry lane is computed as follows:

$$d_{NB} = \frac{3{,}600}{497} + 900(0.25)\left[0.85 - 1 + \sqrt{(0.85-1)^2 + \frac{\left(\frac{3{,}600}{497}\right)0.85}{450(0.25)}}\right]$$

$$+ 5 \times \min[0.85, 1] = 39.6 \text{ s/veh (assuming no rounding of } x)$$

Similarly, d_{SB} = 19.9 s, $d_{\text{bypass},SB}$ = 0 s (assumed), d_{EB} = 46.7 s, d_{WB} = 56.5 s, and $d_{\text{bypass},WB}$ = 41.5 s.

Step 10: Determine LOS for Each Lane on Each Approach

Using Exhibit 21-1, the LOS for each lane is determined as follows:

Lane	Control Delay (s/veh)	LOS
Northbound entry	39.6	E
Southbound entry	19.9	C
Southbound bypass lane	0 (assumed)	A
Eastbound entry	46.7	E
Westbound entry	56.5	F
Westbound bypass lane	41.5	E

Step 11: Compute the Average Control Delay and Determine LOS for Each Approach and the Roundabout as a Whole

The control delays for the northbound and eastbound approaches are equal to the control delay for the entry lanes, as both of these approaches have only one lane. On the basis of Exhibit 21-1, these approaches are both assigned LOS E.

The control delay calculations for the westbound and southbound approaches include the effects of their bypass lanes as follows:

$$d_{WB} = \frac{(56.5)(557) + (41.5)(649)}{557 + 649} = 48.4 \text{ s/veh}$$

$$d_{SB} = \frac{(19.9)(308) + (0.0)(617)}{308 + 617} = 6.6 \text{ s/veh}$$

On the basis of Exhibit 21-1, these approaches are respectively assigned LOS E and LOS A.

Similarly, intersection control delay is computed as follows:

$$d_{\text{intersection}} = \frac{(39.6)(420) + (6.6)(925) + (46.7)(643) + (48.4)(1206)}{420 + 925 + 643 + 1206} = 34.8 \text{ s/veh}$$

On the basis of Exhibit 21-1, the intersection is assigned LOS D.

Step 12: Compute 95th Percentile Queues for Each Lane

The 95th percentile queue is computed for each lane. An example calculation for the northbound entry is given as follows:

$$Q_{95,NB} = 900(0.25)\left[0.85 - 1 + \sqrt{(1-0.85)^2 + \frac{\left(\frac{3{,}600}{497}\right)0.85}{150(0.25)}}\right]\left(\frac{497}{3{,}600}\right) = 8.6 \text{ veh}$$

For design purposes, this value is typically rounded up to the nearest vehicle, which for this case would be 9 veh.

Similarly, $Q_{95,SB}$ = 3.9 veh, $Q_{95,EB}$ = 13.4 veh, $Q_{95,WB}$ = 13.3 veh, and $Q_{95,\text{bypass},WB}$ = 12.5 veh.

Discussion

The results indicate that the overall roundabout is operating at LOS D based on a control delay very close to the boundary between LOS D and LOS E. However, three approaches (northbound, eastbound, and westbound) are operating at LOS E, and one lane (westbound entry) is operating at LOS F (based on control delay). In addition, two of the four entries have volume-to-capacity ratios exceeding 0.95 during the peak 15 min of the hour analyzed. If the performance standard for this intersection were LOS D, three approaches would not meet the standard, even though the overall intersection meets the standard. For these reasons, the analyst should report volume-to-capacity ratios, control delay, and queue lengths for each lane, in addition to the aggregated measures, for a more complete picture of operational performance.

The analyst should be careful not to mask key operational performance issues by reporting overall intersection performance without also reporting the performance of each lane, or at least the worst-performing lane.

EXAMPLE PROBLEM 2: MULTILANE ROUNDABOUT

This is also an example of an operational analysis, despite the fact that lane utilization data are unknown and must be assumed.

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Percent heavy vehicles for eastbound and westbound movements = 5%,
- Percent heavy vehicles for northbound and southbound movements = 2%,
- Peak hour factor = 0.95,
- Negligible pedestrian activity, and
- Volumes and lane configurations as shown in Exhibit 21-24.

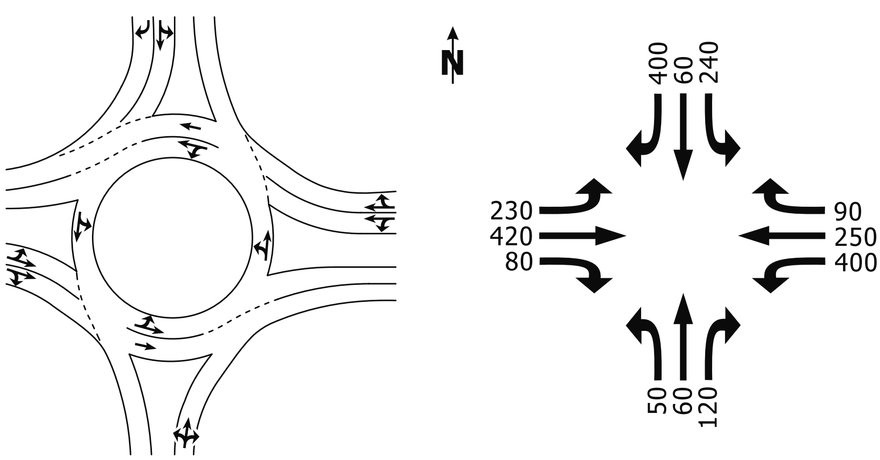


Exhibit 21-24
Demand Volumes and Lane Configurations for Example Problem 2

Comments

Lane use is not specified for the eastbound and westbound approaches; therefore, the percentage flow in the right lane is assumed to be 53%, per Exhibit 21-21.

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement demand volume given in the problem is converted to a demand flow rate by dividing by the peak hour factor. As an example, the eastbound left demand volume is converted to a demand flow rate as follows:

$$v_{EBL} = \frac{V_{EBL}}{PHF} = \frac{230}{0.95} = 242 \text{ veh/h}$$

Step 2: Adjust Flow Rates for Heavy Vehicles

The heavy-vehicle adjustment factor for the eastbound and westbound movements is calculated as follows:

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right)} = \frac{1}{1 + 0.05(2 - 1)} = 0.952$$

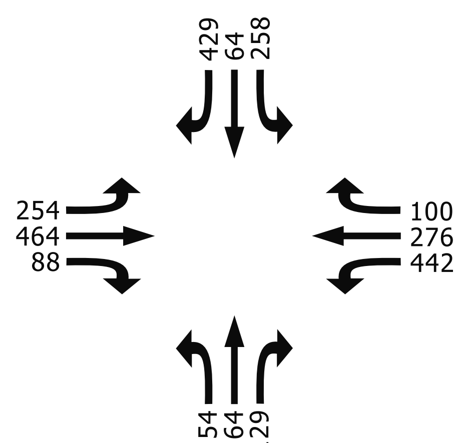
Similarly, the heavy-vehicle adjustment factor for the northbound and southbound movements is calculated as follows:

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right)} = \frac{1}{1 + 0.02(2 - 1)} = 0.980$$

This is applied to each movement as follows (eastbound left turn illustrated):

$$v_{EBL,pce} = \frac{v_{EBL}}{f_{HV}} = \frac{242}{0.952} = 254 \text{ pc/h}$$

The resulting adjusted flow rates for all movements, accounting for Steps 1 and 2, are therefore as follows:



Step 3: Determine Circulating and Exiting Flow Rates

For this problem, only circulating flows need to be calculated for each leg. For the west leg (eastbound entry), the circulating flow is calculated as follows:

$$v_{c,EB,pce} = v_{NBU,pce} + v_{WBL,pce} + v_{WBU,pce} + v_{SBT,pce} + v_{SBL,pce} + v_{SBU,pce}$$

$$v_{c,EB,pce} = 0 + 442 + 0 + 64 + 258 + 0 = 764 \text{ pc/h}$$

Similarly, $v_{c,WB,pce}$ = 372 pc/h, $v_{c,NB,pce}$ = 976 pc/h, and $v_{c,SB,pce}$ = 772 pc/h.

Step 4: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing up the movement flow rates that enter the roundabout. This problem presents four unique cases.

- *Northbound*: The northbound entry has only one lane. Therefore, the entry flow is simply the sum of the movements, or 54 + 64 + 129 = 247 pc/h.
- *Southbound*: The southbound entry has two lanes: a shared through–left lane and a right-turn-only lane. Therefore, the flow rate in the right lane is simply the right-turn movement flow, or 429 pc/h, and the flow rate in the left lane is the sum of the left-turn and through movements, or 258 + 64 = 322 pc/h.
- *Eastbound*: The eastbound entry has shared left–through and through–right lanes. A check is needed to determine whether any de facto lanes are in effect. These checks are as follows:
  - *Left lane*: The left-turn flow rate, 254 pc/h, is less than the sum of the through and right-turn flow rates, 464 + 88 = 552 pc/h. Therefore, some of the through volume is assumed to use the left lane, and no de facto left-turn lane condition is present.
  - *Right lane*: The right-turn flow rate, 88 pc/h, is less than the sum of the left-turn and through flow rates, 254 + 464 = 718 pc/h. Therefore, some of the through volume is assumed to use the right lane, and no de facto right-turn lane condition is present.
  - The total entry flow (254 + 464 + 88 = 806 pc/h) is therefore distributed over the two lanes, with flow biased to the right lane using the assumed lane-use factor identified previously:
    - Right lane: (806)(0.53) = 427 pc/h
    - Left lane: 806 – 427 = 379 pc/h
- *Westbound*: The westbound entry also has shared left–through and through–right lanes, and so a similar check is needed for de facto lanes. The left-turn flow rate, 442 pc/h, is greater than the sum of the through and right-turn flow rates, 276 + 100 = 376 pc/h. Therefore, the left lane is assumed to operate as a de facto left-turn lane. Therefore, the left-lane flow rate is equal to the left-turn flow rate, or 442 pc/h, and the right-lane flow rate is equal to the sum of the through- and right-turn-movement flow rates, or 376 pc/h.

Step 5: Determine the Capacity of Each Entry Lane and Bypass Lane as Appropriate in Passenger Car Equivalents

The capacity calculations for each approach are calculated as follows:

- *Northbound*: The northbound entry is a single-lane entry opposed by two circulating lanes. Therefore, Equation 21-3 is used as follows:

$$c_{pce,NB} = 1{,}130e^{\left(-0.7\times10^{-3}\right)(976)} = 571 \text{ pc/h}$$

- *Southbound*: The southbound entry is a two-lane entry opposed by two circulating lanes. Therefore, Equation 21-4 is used for the right lane, and Equation 21-5 is used for the left lane:

$$c_{pce,SB,R} = 1{,}130e^{\left(-0.7\times10^{-3}\right)(772)} = 658 \text{ pc/h}$$

$$c_{pce,SB,L} = 1{,}130e^{\left(-0.75\times10^{-3}\right)(772)} = 633 \text{ pc/h}$$

- *Eastbound*: The eastbound entry is a two-lane entry opposed by one circulating lane. Therefore, the capacity for each lane is calculated by using Equation 21-2 as follows:

$$c_{pce,EB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(764)} = 526 \text{ pc/h}$$

- *Westbound*: The westbound entry is also a two-lane entry opposed by one circulating lane, so its capacity calculation is similar to that for the eastbound entry:

$$c_{pce,WB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(372)} = 779 \text{ pc/h}$$

Step 6: Determine Pedestrian Impedance to Vehicles

For this problem pedestrians have been assumed to be negligible, so no impedance calculations are performed.

Step 7: Convert Lane Flow Rates and Capacities into Vehicles per Hour

The capacity for a given lane is converted back to vehicles by first determining the heavy-vehicle adjustment factor for the lane and then multiplying it by the capacity in passenger car equivalents. For this example, since all turning movements on the eastbound and westbound entries have the same f_{HV}, each of the lanes on the eastbound and westbound entries can be assumed to have the same f_{HV}, 0.952.

$$c_{EB,R} = c_{pce,EB,R} f_{HV,e,EB} = (526)(0.952) = 501 \text{ veh/h}$$

Similarly, $c_{EB,L}$ = 501 veh/h, $c_{WB,L}$ = 742 veh/h, and $c_{WB,R}$ = 742 veh/h.

Since all turning movements on the northbound and southbound entries have the same f_{HV}, each of the lanes on those entries can be assumed to have the same f_{HV}, 0.980.

$$c_{NB} = c_{pce,NB} f_{HV,e,NB} = (571)(0.980) = 560 \text{ veh/h}$$

Similarly, $c_{SB,L}$ = 621 veh/h and $c_{SB,R}$ = 645 veh/h.

Calculations for the entry flow rates are as follows:

$$v_{EB,R} = v_{pce,EB,R} f_{HV,e,EB} = (427)(0.952) = 407 \text{ veh/h}$$

$$v_{NB} = v_{pce,NB} f_{HV,e,NB} = (247)(0.980) = 242 \text{ veh/h}$$

Similarly, $v_{EB,L}$ = 361 veh/h, $v_{WB,L}$ = 421 veh/h, $v_{WB,R}$ = 358 veh/h, $v_{SB,L}$ = 316 veh/h, and $v_{SB,R}$ = 421 veh/h.

Step 8: Compute the Volume-to-Capacity Ratio for Each Lane

The volume-to-capacity ratio for each lane is calculated as follows:

$$x_{NB} = 242/560 = 0.43$$

$$x_{SB,L} = 316/621 = 0.51$$

$$x_{SB,R} = 421/645 = 0.65$$

$$x_{EB,L} = 361/501 = 0.72$$

$$x_{EB,R} = 407/501 = 0.81$$

$$x_{WB,L} = 421/742 = 0.57$$

$$x_{WB,R} = 358/742 = 0.48$$

Step 9: Compute the Average Control Delay for Each Lane

The control delay for the northbound entry lane is computed as follows:

$$d_{NB} = \frac{3{,}600}{560} + 900(0.25)\left[\frac{242}{560} - 1 + \sqrt{\left(\frac{242}{560} - 1\right)^2 + \frac{\left(\frac{3{,}600}{560}\right)\frac{242}{560}}{450(0.25)}}\right] + 5 \times \min\left[\frac{242}{560}, 1\right] = 13.4 \text{ s/veh}$$

Similarly, $d_{SB,L}$ = 14.2 s, $d_{SB,R}$ = 18.7 s, $d_{EB,L}$ = 27.2 s, $d_{EB,R}$ = 35.4 s, $d_{WB,L}$ = 13.9 s, and $d_{WB,R}$ = 11.7 s.

Step 10: Determine LOS for Each Lane on Each Approach

On the basis of Exhibit 21-1, the LOS for each lane is determined as follows:

Critical Lane	Control Delay (s/veh)	LOS
Northbound entry	13.4	B
Southbound left lane	14.2	B
Southbound right lane	18.7	C
Eastbound left lane	27.2	D
Eastbound right lane	35.4	E
Westbound left lane	13.9	B
Westbound right lane	11.7	B

Step 11: Compute the Average Control Delay and Determine LOS for Each Approach and the Roundabout as a Whole

The control delay for the northbound approaches is equal to the control delay for the entry lane, 13.4 s, as the approach has only one lane. The control delays for the other approaches are as follows:

$$d_{SB} = \frac{(14.2)(316) + (18.7)(421)}{316 + 421} = 16.8 \text{ s/veh}$$

$$d_{EB} = \frac{(27.2)(361) + (35.4)(407)}{361 + 407} = 31.5 \text{ s/veh}$$

$$d_{WB} = \frac{(13.9)(421) + (11.7)(358)}{421 + 358} = 12.9 \text{ s/veh}$$

On the basis of Exhibit 21-1, these approaches are respectively assigned LOS B, LOS C, LOS D, and LOS B.

Similarly, control delay for the intersection is computed as follows:

$$d_{\text{intersection}} = \frac{(13.4)(242) + (16.8)(736) + (31.5)(768) + (12.9)(779)}{242 + 736 + 768 + 779} = 19.7 \text{ s/veh}$$

On the basis of Exhibit 21-1, the intersection is assigned LOS C.

Step 12: Compute 95th Percentile Queues for Each Lane

The 95th percentile queue is computed for each lane. An example calculation for the northbound entry is given as follows:

$$Q_{95,NB} = 900(0.25)\left[\frac{242}{560} - 1 + \sqrt{\left(1 - \frac{242}{560}\right)^2 + \frac{\left(\frac{3{,}600}{560}\right)\left(\frac{242}{560}\right)}{150(0.25)}}\right]\left(\frac{560}{3{,}600}\right) = 2.2 \text{ veh}$$

For design purposes, this value is typically rounded up to the nearest vehicle, in this case 3 veh.

Discussion

The results indicate that the intersection as a whole operates at LOS C on the basis of control delay during the peak 15 min of the analysis hour. However, the eastbound approach operates at LOS D, and the right lane of that approach operates at LOS E (with a control delay very close to the boundary of LOS D and LOS E) and with a volume-to-capacity ratio of 0.81. The analyst should report both the overall performance and those of the individual lanes to provide a more complete picture of operational performance.

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Exhibit 22-47
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-2Q Interchanges

Input		Output				
			Intersection I		**Intersection II**	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		EXT-LT = G	
E			RT		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT(I) = C		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-48
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-4Q Interchanges

Input		Output				
			Intersection I		**Intersection II**	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-49
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-2Q Interchanges

Input		Output				
			Intersection I		**Intersection II**	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT = C		LT	
H			RT = D+N		RT	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT = B	
L			RT		RT = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-50
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT(II) = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-51
Worksheet for Obtaining Turning Movements from O-D Movements for Diamond Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH = K	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH = L		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-52
Worksheet for Obtaining Turning Movements from O-D Movements for SPUIs

Input		Output		
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT = E	
B			RT = F	
C			TH = I	
D		Westbound (WB)	LT = H	
E			RT = G	
F			TH = J	
G		Northbound (NB)	LT = A	
H			RT = B	
I			TH = K	
J			UT	
K		Southbound (SB)	LT = D	
L			RT = C	
M			TH = L	
N			UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for the application of alternative tools to the analysis of interchange ramp terminals. Chapter 34, Interchange Ramp Terminals: Supplemental, contains supplemental examples illustrating the use of alternative tools for interchange analysis. Additional information on this topic may be found in the Technical Reference Library in Volume 4.

General guidance on alternative tools is provided in Chapters 6 and 7.

As indicated in Chapter 6, traffic models may be classified in several ways (e.g., deterministic versus stochastic, macroscopic versus microscopic). The alternative tools used for interchange analysis are generally based on models that are microscopic and stochastic in nature. Therefore, the discussion in this section will be limited to microsimulation tools.

Strengths of the HCM Procedure

This chapter offers a comprehensive procedure for analyzing the performance of several types of interchanges. Simulation-based tools offer a more detailed treatment of the arrival and departure of individual vehicles and features of the signal control system, but for most purposes, the HCM procedure produces an acceptable approximation. The HCM procedure offers some advantages over the simulation approach:

- The HCM provides saturation flow rate adjustment factors based on extensive field studies.
- The HCM produces direct estimates of capacity and *v/c* ratio. These measures are much more elusive in simulation.
- The HCM provides LOS by O-D, which facilitates the comparison of operational performance for different interchange configurations.
- It provides deterministic estimates of the measures of effectiveness, which is important for some purposes such as development impact review.
- Simulation tools use definitions of delay (and therefore LOS) different from those of the HCM, especially for movements that are oversaturated at some point during the analysis. Great care must therefore be taken in producing LOS estimates directly from simulation. Chapter 7 discusses simulation-based performance measures in more detail.

Identified Limitations of the HCM Procedure

The identified limitations of the HCM procedure for this chapter cover a number of conditions that are not evaluated explicitly, including the following:

- Oversaturated conditions, particularly cases when the downstream queue spills back into the upstream intersection for long periods of time;
- The impact of spillback on freeway operations (however, the method does estimate the expected queue storage ratio for the ramp approaches);
- Ramp metering and its resulting spillback of vehicles into the interchange;

- Impacts of the interchange operations on arterial operations and the extended surface street network;
- Lane utilizations for interchanges with additional approaches that are not part of the prescribed interchange configuration; and
- Full cloverleaf interchanges (freeway-to-freeway or system interchanges), since the scope of the chapter is limited to service interchanges (e.g., freeway-to-arterial interchanges).

If any of these conditions apply to a particular situation, then alternative tools that recognize them explicitly should be considered to supplement or replace the methodology described in this chapter.

Additional Features and Performance Measures Available From Alternative Tools

This chapter provides a methodology for estimating the capacity, control delay, queue storage ratio, and LOS associated with a given set of traffic, control, and design conditions at an interchange. As with most other procedural chapters in this manual, simulation outputs, especially graphics-based presentations, can provide details on problems at specific elements of the interchange that might otherwise go unnoticed with a macroscopic analysis. For example, problems associated with turn bay overflow or blockage of access to turn bays can be better observed by using microscopic simulation tools. Alternative tools offer additional performance measures such as number of stops, fuel consumption, and pollution. The animated graphics displays offered by many simulation tools are especially useful for observing network operations and identifying problems at specific elements.

Development of HCM-Compatible Performance Measures Using Alternative Tools

Simulation tools provide a wealth of information with regard to performance measures, including queue length, travel time, emissions, and so forth. However, simulation tools often have different definitions for each of these performance measures. General guidance on developing compatible performance measures based on the analysis of individual vehicle trajectories is provided in Chapter 7, with supplemental examples provided in Chapter 24, Concepts: Supplemental. Chapter 18, Signalized Intersections, provides some specific guidance on performance measures for signalized approaches that also applies to this chapter. To obtain LOS for a specific O-D, the analyst will need to obtain the performance measures for the specific approaches using that particular O-D and aggregate them as indicated in the methodology section of this chapter.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

For interchanges, the definitions of delay and queuing are the most significant conceptual differences between the HCM and simulation modeling. Both are measures of effectiveness used to obtain LOS for each O-D, and simulated estimates of them would produce results inherently different from those obtained by the analytical method described in this chapter.

Lane utilization is also treated differently. Simulation tools derive lane distributions and utilization implicitly from driver behavior modeling, while the deterministic model used in this chapter develops lane distributions from empirical models. Differences in the treatment of random arrivals are also an issue in the comparison of performance measures. This topic was discussed in detail in Chapter 18, and the same phenomena apply to this chapter.

In some cases, and when saturation flow rate is not an input, simulation tools do not explicitly account for differences between left-turning, through, and right-turning movements, and all three have very similar saturation headway values. Thus, the left- and right-turn lane capacity would likely be overestimated in those types of tools.

Adjustment of Simulation Parameters to Match the HCM Parameters

Some adjustments will generally be required before an alternative tool can be used effectively to supplement or replace the procedures described in this chapter. For example, the parameters that determine the capacity of a signalized approach (e.g., steady state headway and start-up lost time) should be adjusted to ensure that the simulated approach capacities match the HCM values.

One parameter specific to this chapter is the lane utilization on the approaches within the interchange. Driver behavior model parameters that affect lane choice should be examined closely and modified if necessary to produce better agreement with the lane distributions estimated by the procedures in this chapter.

Simulation tools do not produce explicit capacity estimates. The accepted method of determining the capacity of a signalized approach by simulation is to perform the simulation run(s) with a demand in excess of the computed capacity and use the throughput as an indication of capacity. Chapter 7 provides additional guidance on the determination of capacity in this manner. The Chapter 7 discussion points out the complexities that can arise when self-aggravating phenomena occur as the operation approaches capacity. Because of the interaction of traffic movements within an interchange, the potential for self-aggravating situations is especially high.

In complex situations, conceptual differences between the deterministic procedures in this chapter and those of simulation tools may exist such that the production of compatible capacity estimates is not possible. In such cases, the capacity differences should be noted.

Step-by-Step Recommendations for Applying Alternative Tools

General guidance on selecting and applying alternative tools is provided in Chapters 6 and 7. Chapter 18 provides recommendations specifically for signalized intersections that also apply to interchange ramp terminals.

One step that is specific to this chapter is the emulation of the traffic control hardware. Generally, simulation tools provide great flexibility in emulating actuated control, particularly in the type and location of detectors. In most cases, simulation tools attach a controller to each intersection (or node) in the network. This creates problems for some interchange operations in which a controller at

Supplemental problems involving the use of alternative tools for signalized intersection analysis are presented in Chapter 34.

one node must be connected to an approach to another node. A diamond interchange operating with one controller is an example of the complexities that can arise in the emulation of the traffic control scheme.

Some tools are able to accommodate complex schemes more flexibly than others. The ability to emulate the desired traffic control scheme is an important consideration in the selection of a tool for interchange analysis.

Sample Calculations Illustrating Alternative Tool Applications

Example Problem 1 in this chapter involves a diamond interchange that offers the potential for illustrating the use of alternative tools. There are no limitations in this example that suggest the need for alternative tools. It is possible, however, to introduce situations in which alternative tools might be needed for a proper assessment of performance.

Chapter 34 includes supplemental examples that apply alternative tools to deal with two conditions that are beyond the scope of the procedures presented in this chapter.

1. A two-way STOP-controlled intersection in close proximity to the diamond interchange and
2. Ramp metering on one of the freeway entrance ramps connected to the interchange.

In both cases, the demand volumes are varied to examine the self-aggravating effects on the operation of the facility.

4. EXAMPLE PROBLEMS

INTRODUCTION

This part of the chapter describes the application of each of the final design, operational analysis for interchange type selection, and roundabouts analysis methods through the use of example problems. Exhibit 22-53 describes each of the three example problems included in this chapter and indicates the methodology applied. Additional example problems are provided in Chapter 34, Interchange Ramp Terminals: Supplemental.

Exhibit 22-53
Example Problem Descriptions

Problem Number	Description	Application
1	Find the control delay, queue storage ratio, and LOS of a diamond interchange	Operational analysis
2	Find the control delay, queue storage ratio, and LOS of a Parclo A-2Q interchange	Operational analysis
3	Compare eight types of signalized interchanges	Operational analysis for interchange type selection

EXAMPLE PROBLEM 1: DIAMOND INTERCHANGE

The Interchange

The interchange of I-99 (northbound/southbound, NB/SB) and University Drive (eastbound/westbound, EB/WB) is a diamond interchange. Exhibit 22-54 provides the interchange volumes and channelization, while Exhibit 22-55 provides the signalization information.

Exhibit 22-54
Example Problem 1: Interchange Volumes and Channelization

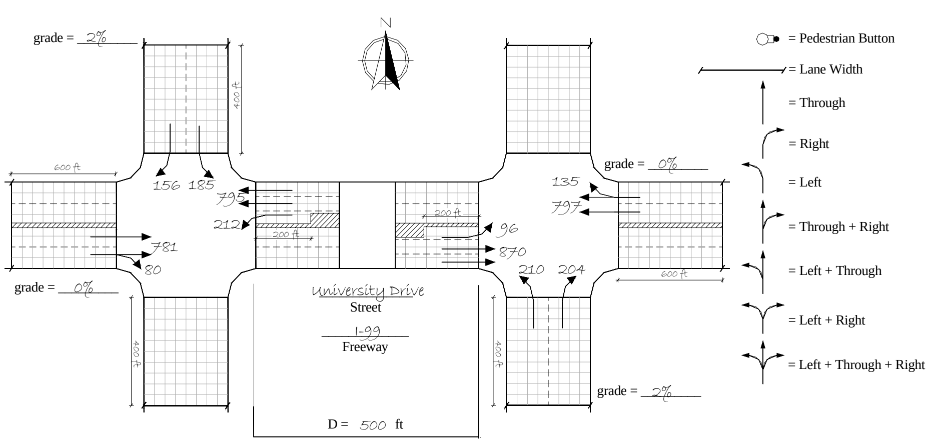


Exhibit 22-55
Example Problem 1: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (4+8)	Φ (4+7)	Φ (2+5)	Φ (4+8)	Φ (1+6)	Φ (3+8)
Green time (s)	63	43	39	63	53	29
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		19			9	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 50 ft for all right-turning movements and 75 ft for all left-turning movements. Arrival Type 4 is assumed for all arterial movements and Arrival Type 3 for all other movements.

There are 5% heavy vehicles on both the external and internal through movements, and the peak hour factor (PHF) for the interchange is estimated to be 0.90. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Outline of Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). Since all movements utilize the signal and no right-turns-on-red are allowed, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 22-56.

Exhibit 22-56
Example Problem 1:
Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	210	233
B	204	227
C	156	173
D	185	206
E	96	107
F	80	89
G	135	150
H	212	236
I	685	761
J	585	650
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

Both external approaches to this interchange consist of a two-lane shared right and through lane group. Use of the two-lane model of Exhibit 22-16 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 22-57.

Exhibit 22-57
Example Problem 1: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	Maximum Lane Utilization	Lane Utilization Factor
Eastbound external	0.5056	0.4944	0.5056	0.9890
Westbound external	0.5181	0.4819	0.5181	0.9651

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 22-57. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated as indicated in Chapter 18, Signalized Intersections. The estimated saturation flow rates for the eastbound approaches are shown in Exhibit 22-58.

Calculation of Common Green and Lost Time due to Downstream Queue and Demand Starvation

The duration of common green between various movements is calculated next. Exhibit 22-59 provides the beginning and end of each phase for the two intersections, as well as the calculations of common green between various movements at the two intersections.

Exhibit 22-58
Example Problem 1: Calculation of Saturation Flow Rate for Eastbound Approaches

	Eastbound Turning Movements		
	EXT-TH, EXT-RT	INT-TH	INT-LT
Base saturation flow (s_0, in pc/hg/ln)	1,900	1,900	1,900
Number of lanes (N)	2	2	1
Lane width adjustment (f_w)	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.989	0.952	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	0.999	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.991	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	1.034	1.036	0.963
Adjusted saturation flow (s, in veh/hg/ln)	3,700	3,568	1,703

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-59
Example Problem 1:
Common Green Calculations

Phase	Intersection I Phase Begin	Intersection I Phase End	Intersection II Phase Begin	Intersection II Phase End	
Phase 1	0	63	150	53	
Phase 2	68	111	58	111	
Phase 3	116	155	116	145	
Movement	**1st Green Time of Cycle Begin**	**1st Green Time of Cycle End**	**2nd Green Time of Cycle Begin**	**2nd Green Time of Cycle End**	**Overlap**
EB EXT-TH	0	63			53
EB INT-TH	150	53	116	150	
WB EXT-TH	150	53			53
WB INT-TH	0	111			
SB RAMP	116	155			34
EB INT-TH	150	53	116	150	
NB RAMP	58	111			53
WB INT-TH	0	111			
WB INT-LT	68	111			0
EB INT-TH	150	53			
EB INT-LT	116	145			0
WB INT-TH	0	111			

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, INT = internal, EXT = external.

The next step involves the calculation of lost time due to downstream queues. First, the queue at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. These numbers are then subtracted from the internal link storage length to determine the storage available at the beginning of the respective upstream phase. Exhibit 22-60 presents the calculation of the downstream queues followed by the calculation of the respective lost time due to those downstream queues.

Exhibit 22-60
Example Problem 1:
Calculation of Lost Time due to Downstream Queues

	EB EXT-TH	SB-LT	WB EXT-TH	NB-LT
V_R or V_A (veh/h)	206	868	233	886
N_R or N_A	1	2	1	2
G_R or G_A (s)	39	63	53	63
G_D (s)	97	97	111	111
C (s)	160	160	160	160
CG_{UD} or CG_{RD} (s)	53	34	53	53
Queue length (Q_A or Q_R) (ft)	0.0	4.1	0.0	0.0
G_R or G_A (s)	63	39	63	53
C (s)	160	160	160	160
D_{QA} or D_{QR} (ft)	500	496	500	500
CG_{UD} or CG_{RD} (s)	53	34	53	53
Additional lost time ($L_{D\text{-}A}$ or $L_{D\text{-}R}$) (s)	0.0	0.0	0.0	0.0
Total lost time t'_L (s)	5.0	5.0	5.0	5.0
Effective green time g' (s)	63.0	39.0	63.0	53.0

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, EXT = external.

Calculation of lost time due to demand starvation begins by determining the queue storage length at the beginning of an interval with demand starvation potential by using Equation 22-17. The lost time due to demand starvation is then calculated by using Equation 22-16. The respective calculations are presented in Exhibit 22-61.

Exhibit 22-61
Example Problem 1: Calculation of Lost Time due to Demand Starvation

	EB INT-TH	WB INT-TH
$v_{Ramp\text{-}L}$ (veh/h)	206	233
$v_{Arterial}$ (veh/h)	868	886
C (s)	160	160
$N_{Ramp\text{-}L}$	1	1
$N_{Arterial}$	2	2
CG_{RD} (s)	34	53
CG_{UD} (s)	53	53
H_I	2.02	2.04
$Q_{INITIAL}$ (ft)	0	0
CG_{DS} (s)	0	0
L_{DS} (s)	0	0
t''_L (s)	5	5
Effective green time g'' (s)	97	111

Note: EB = eastbound, WB = westbound, TH = through, INT = internal.

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 22-62 presents the calculation of the queue storage ratio for all eastbound movements in Example Problem 1. Control delay for each movement is calculated according to Equation 18-47. Exhibit 22-63 provides the control delay for each eastbound movement of the interchange.

Exhibit 22-62
Example Problem 1: Queue Storage Ratio Calculations for Eastbound Movements

	EXT-TH, EXT-RT	INT-LT	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0
v (veh/h/ln group)	957	107	967
s (veh/h/ln)	1850	1703	1784
g (s)	63	29	97
g/C	0.39	0.18	0.61
I	1.00	0.71	0.71
c (veh/h/ln group)	1459	309	2163
$X = v/c$	0.66	0.35	0.45
r_a (ft/s^2)	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0
S_s (mi/h)	5	5	5
S_{pl} (mi/h)	40	40	40
S_a (mi/h)	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04
Rp	1.000	1.333	1.333
P	0.39	0.24	0.81
r (s)	97	131	63
t_f (s)	0.01	0.00	0.00
q (veh/s)	0.27	0.03	0.27
q_g (veh/s)	0.27	0.04	0.36
q_r (veh/s)	0.27	0.03	0.13
Q_1 (veh)	15.22	3.47	3.84
Q_2 (veh)	0.93	0.14	1.37
T	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00
t_A	0	0	0
Q_e (veh)	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00
Q_3 (veh)	0.00	0.00	0.00
Q (veh)	16.15	3.65	3.98
L_h (ft)	25.01	25.00	25.01
L_a (ft)	600	200	500
R_Q	0.67	0.46	0.20

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-63
Example Problem 1: Calculation of Control Delay for Eastbound Movements

	EXT-TH, EXT-RT	INT-LT	INT-TH
g (s)	NA	29	97
g' (s)	63	NA	NA
g/C or g'/C	0.39	0.18	0.61
c (veh/h)	1459	309	2163
$X = v/c$	0.66	0.35	0.45
d_1 (s/veh)	39.6	52.8	7.3
k	0.5	0.5	0.5
d_2 (s/veh)	4.6	2.2	0.5
d_3 (s/veh)	0.0	0.0	0.0
k_{min}	0.04	0.04	0.04
u	0	0	0
t	0	0	0
d (s/veh)	44.1	55.0	7.8

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external, NA = not applicable.

Results

This section discusses the estimation of the delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as a sum of the movement delays for each movement utilized by the O-D, as shown in Equation 22-1. Next, the v/c and queue storage ratios must be checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement will be F. Exhibit 22-64 presents a summary of the results for all O-D movements at this interchange. As shown, neither v/c nor R_Q exceed 1, and all movements have LOS E or better. The LOS is determined by using Exhibit 22-11.

Exhibit 22-64
Example Problem 1: O-D Movement LOS

O-D Movement	Delay (s)	v/c > 1?	R_Q > 1?	LOS
A	45.6	No	No	C
B	43.7	No	No	C
C	54.6	No	No	C
D	63.6	No	No	D
E	99.2	No	No	E
F	44.2	No	No	C
G	37.5	No	No	C
H	82.7	No	No	D
I	52.0	No	No	C
J	39.8	No	No	C

EXAMPLE PROBLEM 2: PARCLO A-2Q INTERCHANGE

The Interchange

The interchange of I-75 (NB/SB) and Newberry Avenue (EB/WB) is a Parclo A-2Q interchange. Exhibit 22-65 provides the interchange volumes and channelization, while Exhibit 22-66 provides the signalization information.

Exhibit 22-65
Example Problem 2: Intersection Plan View

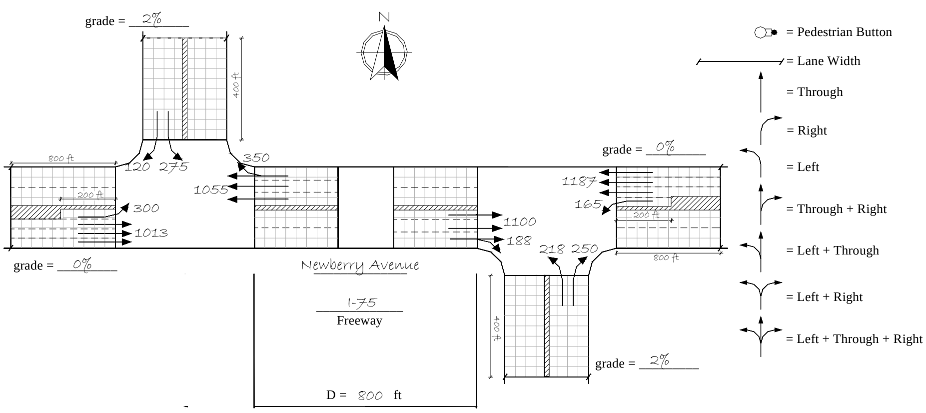


Exhibit 22-66
Example Problem 2: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (3+8)	Φ (4+8)	Φ (2+5)	Φ (4+7)	Φ (1+6)	Φ (4+8)
Green time (s)	25	60	40	25	35	65
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		0			0	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. The eastbound and westbound left-turn radii are 80 ft, while all remaining turning movements have radii of 50 ft. The arrival type is assumed to be 4 for arterial movements and 3 for all other movements.

There are 10% heavy vehicles on both the external and internal through movements, and the PHF for the interchange is estimated to be 0.95. Start-up lost time is 3 s for all approaches, while the extension of effective green is 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Outline of Solution

Calculation of O-Ds

O-Ds through this interchange are calculated on the basis of Exhibit 22-21(b). Since all movements utilize the signal and no right-turns-on-red are allowed,

O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 22-67.

Exhibit 22-67 Example Problem 2: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	218	229
B	250	263
C	120	126
D	275	289
E	188	198
F	300	316
G	165	174
H	350	368
I	825	868
J	837	881
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

The external approaches to this interchange consist of a three-lane through lane group for the external approaches. Use of the three-lane model from Exhibit 22-17 results in the predicted lane utilization percentages for the external through approaches presented in Exhibit 22-68.

Exhibit 22-68 Example Problem 2: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	V_3	Maximum Lane Utilization	Lane Utilization Factor
Eastbound	0.2660	0.2791	0.4549	0.4549	0.7328
Westbound	0.2263	0.2472	0.5265	0.5265	0.6332

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the external approaches is obtained as shown above in Exhibit 22-68. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated according to Chapter 18. The results of these calculations for the eastbound approaches are presented in Exhibit 22-69.

Exhibit 22-69
Example Problem 2: Calculation of Saturation Flow Rates for Eastbound Approaches

	EXT-TH	EXT-LT	INT-TH, INT-RT
Base saturation flow (s_0, in pc/hg/ln)	1,900	1,900	1,900
Number of lanes (N)	3	1	3
Lane width adjustment (f_w)	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.909	1.000	0.909
Grade adjustment (f_g)	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.733	1.000	1.000
Left-turn adjustment (f_{LT})	1.000	0.934	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	0.998
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	1.000	0.934	0.985
Traffic pressure adjustment for lane group (f_v)	0.997	1.013	1.016
Adjusted saturation flow (s, in veh/hg/ln)	3786	1798	5253

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Calculation of Common Green and Lost Time due to Downstream Queue

The duration of common green between various movements is calculated next. Exhibit 22-70 presents the beginning and ending of each phase at the two intersections and the calculations of common green between various movements at the two intersections.

Exhibit 22-70
Example Problem 2: Common Green Calculations

	Intersection I		Intersection II		
Phase	**Phase Begin**	**Phase End**	**Phase Begin**	**Phase End**	
Phase 1	0	25	0	25	
Phase 2	30	90	30	65	
Phase 3	95	135	70	135	
	1st Green Time of Cycle		**2nd Green Time of Cycle**		
Movement	**Begin**	**End**	**Begin**	**End**	**Overlap**
EB EXT-TH	0	90			20
EB INT-TH	70	135			
WB EXT-TH	0	25	70	135	20
WB INT-TH	30	90			
SB RAMP	95	135			40
EB INT-TH	70	135			
NB RAMP	30	65			35
WB INT-TH	30	90			

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, TH = through, INT = internal, EXT = external.

The next step involves the calculation of lost time due to downstream queues. First, the queue at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. These numbers are then subtracted from the internal link storage length to determine the storage at the beginning of the respective upstream phase. Exhibit 22-71 presents the calculation of the downstream queues followed by the calculation of the respective lost time due to those downstream queues.

Exhibit 22-71
Example Problem 2: Calculation of Lost Time due to Downstream Queues

	EB EXT-TH	SB-LT	WB EXT-TH	NB-LT
V_R or V_A (veh/h)	289	1066	229	1249
N_R or N_A	1	3	1	3
G_R or G_A (s)	40	90	35	95
G_D (s)	65	65	60	60
C (s)	140	140	140	140
CG_{UD} or CG_{RD} (s)	20	40	20	35
Queue length (Q_A or Q_R) (ft)	0.9	48.6	0.0	89.4
G_R or G_A (s)	90	40	95	35
C (s)	140	140	140	140
D_{QA} or D_{QR} (ft)	799	751	800	711
CG_{UD} or CG_{RD} (s)	20	40	20	35
Additional lost time ($L_{D\text{-}A}$ or $L_{D\text{-}R}$) (s)	0	0	0	0
Total lost time t'_L (s)	6	6	6	6
Effective green time g' (s)	89	39	94	34

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, EXT = external.

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 22-72 presents the calculation of queue storage for all eastbound movements. Control delay for each movement is calculated according to Equation 18-47. Exhibit 22-73 provides the control delay for each eastbound movement of this interchange.

Exhibit 22-72
Example Problem 2: Queue Storage Ratio Calculations for Eastbound Movements

	EXT-TH	EXT-LT	INT-TH, INT-RT
Q_{bL} (ft)	0.0	0.0	0.0
v (veh/h/ln group)	1066	316	1282
s (veh/h/ln)	1262	1798	1751
g (s)	89	24	64
g/C	0.64	0.17	0.46
I	1.00	1.00	0.90
c (veh/h/ln group)	2407	308	2401
$X = v/c$	0.44	1.02	0.54
r_a (ft/s^2)	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0
S_s (mi/h)	5	5	5
S_{pl} (mi/h)	40	40	40
S_a (mi/h)	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04
Rp	1.000	1.000	1.333
P	0.64	0.17	0.61
r (s)	51	116	76
t_f (s)	0.00	0.01	0.00
q (veh/s)	0.30	0.09	0.38
q_g (veh/s)	0.30	0.09	0.50
q_r (veh/s)	0.30	0.09	0.27
Q_1 (veh)	5.36	10.75	6.91
Q_2 (veh)	0.13	4.94	0.33
T	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00
t_A	0	0	0
Q_e (veh)	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00
Q_3 (veh)	0.00	0.00	0.00
Q (veh)	5.49	15.69	7.24
L_h (ft)	25.02	25.00	25.02
L_a (ft)	800	200	800
R_Q	0.17	1.96	0.23

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-73
Example Problem 2: Control Delay for Eastbound Movements

	EXT-TH	INT-LT	INT-TH, INT-RT
g (s)	NA	24	64
g' (s)	89	NA	NA
g/C or g'/C	0.64	0.17	0.46
c (veh/h)	2407	308	2401
$X = v/c$	0.44	1.02	0.56
d_1 (s/veh)	12.9	58.0	18.81
k	0.5	0.5	0.5
d_2 (s/veh)	0.6	57.7	1.53
d_3 (s/veh)	0.0	0.0	0.0
k_{min}	0.04	0.04	0.04
u	0	0	0
t	0	0	0
d (s/veh)	13.5	115.7	20.34

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external, NA = not applicable.

Results

This section discusses the estimation of delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as a sum of the movement delays for each movement utilized by the O-D, as shown in Equation 22-1. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement will be F. Exhibit 22-74 presents the resulting delay, v/c, and R_Q for each O-D movement. As shown, O-D Movement F has a v/c and R_Q ratio greater than 1, resulting in a LOS of F.

Exhibit 22-74
Example Problem 2: O-D Movement LOS

O-D Movement	Delay (s)	v/c > 1?	R_Q > 1?	LOS
A	78.9	No	No	D
B	55.7	No	No	D
C	41.1	No	No	C
D	70.0	No	No	D
E	33.9	No	No	C
F	115.7	Yes	Yes	F
G	61.6	No	No	D
H	40.0	No	No	C
I	33.9	No	No	C
J	40.0	No	No	C

EXAMPLE PROBLEM 3: OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

The Interchange

An interchange is to be built at the junction of I-83 (NB/SB) and Archer Road (EB/WB) in an urban area. The interchange type selection methodology is used.

The Question

Which interchange type is likely to operate better under the given demands?

The Facts

This interchange will have two-lane approaches with single left-turn lanes on the arterial approaches. Freeway ramps will consist of two-lane approaches with channelized right turns in addition to the main ramp lanes. Default saturation flow rates for use in the interchange type selection analysis are given in Exhibit 22-28. The O-Ds of traffic through this interchange are shown in Exhibit 22-75.

Exhibit 22-75
Example Problem 3: O-D Demand Information for the Interchange

O-D Movement	Volume (veh/h)
A	400
B	350
C	400
D	550
E	150
F	200
G	225
H	185
I	600
J	800
K	2,500
L	3,200
M	0
N	10

Outline of Solution

Mapping O-D Flows into Interchange Movements

The primary objective of this example is to compare up to eight interchange types against a given set of design volumes. The first step is to convert these O-D flows into movement flows through the signalized interchange. The interchange type methodology uses the standard NEMA numbering sequence for interchange phasing, and Exhibit 22-29 demonstrates which O-Ds make up each NEMA phase at the eight interchange types. Exhibit 22-76 shows the corresponding volumes for this example on the basis of the O-Ds from Exhibit 22-75. Since this interchange has channelized right turns, Exhibit 22-77 shows only the NEMA phasing volumes utilizing the signals.

Exhibit 22-76
Example Problem 3: NEMA Flows (veh/h) for the Interchange

Interchange	NEMA Phase Movement Number							
Type	1	2	3	4	5	6	7	8
SPUI	185	800	400	400	150	1,025	560	350
TUDI /CUDI	185	950	--	960	160	1,210	--	750
CDI (I)	185	950	--	960	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	960	--	1,385	--	--
Parclo A-4Q (II)	--	1,310	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	960	200	1,385	--	--
Parclo A-2Q (II)	225	1,310	--	--	--	985	--	750
Parclo B-4Q (I)	185	950	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	950	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Note: (I) and (II) indicate the intersections within the interchange type; -- indicates that the movement does not exist in this interchange type.

Exhibit 22-77
Example Problem 3: NEMA Flows for the Interchange Without Channelized Right Turns

Interchange Type	NEMA Phase Movement Number 1	2	3	4	5	6	7	8
SPUI	185	600	400	0	150	1,025	560	350
TUDI /CUDI	185	750	--	560	160	1,210	--	750
CDI (I)	185	750	--	560	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	560	--	1,385	--	--
Parclo A-4Q (II)	--	1,150	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	560	200	1,385	--	--
Parclo A-2Q (II)	225	1,150	--	--	--	985	--	750
Parclo B-4Q (I)	185	750	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	750	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Computation of Critical Flow Ratios

Comparison between the eight intersection types begins with computation of the critical flow ratio at each interchange type. The first intersection type to be calculated is the SPUI by using Equation 22-26. On the basis of the default saturation flow rate for a SPUI and the values for the NEMA phases, Exhibit 22-78 shows the output from these calculations for a SPUI.

Exhibit 22-78
Example Problem 3: SPUI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements, A	0.368	0.306
Critical flow ratio for the ramp movements, R	0.350	0.156
Sum of critical flow ratios, Y_c	0.718	0.462

The TUDI critical flow ratios are calculated by using Equation 22-27. Exhibit 22-79 shows these calculations for a 300-ft distance between the two TUDI intersections.

Exhibit 22-79
Example Problem 3: TUDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Effective flow ratio for concurrent phase when dictated by travel time, y_t	0.070	0.070
Effective flow ratio for concurrent Phase 3, y_3	0.070	0.070
Effective flow ratio for concurrent Phase 7, y_7	0.070	0.070
Critical flow ratio for the arterial movements, A	0.461	0.294
Critical flow ratio for the ramp movements, R	0.474	0.315
Sum of critical flow ratios, Y_c	0.935	0.609

The CUDI critical flow ratios are calculated by using Equation 22-28. Exhibit 22-80 shows these calculations for a CUDI with the given O-D flows.

Exhibit 22-80
Example Problem 3: CUDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Flow ratio for Phase 2 with consideration of prepositioning, y_2	0.264	0.208
Flow ratio for Phase 6 with consideration of prepositioning, y_6	0.208	0.208
Critical flow ratio for the arterial movements, A	0.373	0.332
Critical flow ratio for the ramp movements, R	0.267	0.156
Sum of critical flow ratios, Y_c	0.640	0.488

The CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q all use separate controllers. For these interchanges the critical flow ratios are calculated for each intersection, and then the maximum is taken for the overall interchange critical flow ratio. These numbers are all calculated by using Equation 22-29 and the default saturation flows. Exhibit 22-81 through Exhibit 22-85 show the calculations for these interchanges utilizing two controllers.

Exhibit 22-81
Example Problem 3: CDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.655	0.498
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.118
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.486
Maximum sum of critical flow ratios, Y_c	0.655	0.498

Exhibit 22-82
Example Problem 3: Parclo A-4Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.385	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.282
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.667	0.615
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.364	0.333
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.208	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.572	0.444
Maximum sum of critical flow ratios, Y_c	0.667	0.615

Exhibit 22-83
Example Problem 3: Parclo A-2Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.502	0.451
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.784	0.616
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.452
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.563
Maximum sum of critical flow ratios, Y_c	0.784	0.616

Exhibit 22-84
Example Problem 3: Parclo B-4Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.000	0.000
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.373	0.333
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.000	0.000
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.430	0.368
Maximum sum of critical flow ratios, Y_c	0.430	0.368

Exhibit 22-85
Example Problem 3: Parclo B-2Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.111	0.111
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.484	0.444
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.103	0.103
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.533	0.471
Maximum sum of critical flow ratios, Y_c	0.533	0.471

Estimation of Interchange Delay

Estimation of interchange delay is the final step when interchange types are compared. On the basis of the critical flow ratios calculated previously, Exhibit 22-36 can be used to calculate the delay at the eight interchange types. Exhibit 22-86 shows the solutions to these calculations.

Exhibit 22-86
Example Problem 3: Interchange Delay for the Eight Interchange Types

Intersection Type	Interchange Delay d_I (s) Right Turns Signalized	Interchange Delay d_I (s) Right Turns Free or YIELD-controlled
SPUI	62.9	22.0
TUDI	217.7	33.3
CUDI	35.9	27.4
CDI	26.6	21.7
Parclo A-4Q	26.2	21.6
Parclo A-2Q	47.4	29.0
Parclo B-4Q	11.9	11.3
Parclo B-2Q	30.7	29.0

Results

As demonstrated by Exhibit 22-86, a Parclo B-4Q would be the best interchange type to select in terms of operational performance for the given O-D flows at this interchange. For the final interchange type selection, however, additional criteria should be considered, including those related to economic, environmental, and land use concerns.

5. REFERENCES

1. Elefteriadou, L., C. Fang, R. P. Roess, E. Prassas, J. Yeon, X. Cui, A. Kondyli, H. Wang, and J. M. Mason. *Capacity and Quality of Service of Interchange Ramp Terminals.* Final Report, National Cooperative Highway Research Program Project 3-60. Pennsylvania State University, University Park, March 2005.
2. Elefteriadou, L., A. Elias, C. Fang, C. Lu, L. Xie, and B. Martin. *Validation and Enhancement of the Highway Capacity Manual's Interchange Ramp Terminal Methodology.* Final Report, National Cooperative Highway Research Program Project 3-60A. University of Florida, Gainesville, 2009.
3. Messer, C. J., and J. A. Bonneson. *Capacity of Interchange Ramp Terminals.* Final Report, National Cooperative Highway Research Program Project 3-47. Texas A&M Research Foundation, College Station, April 1997.
4. Bonneson, J., K. Zimmerman, and M. Jacobson. *Review and Evaluation of Interchange Ramp Design Considerations for Facilities Without Frontage Roads.* Research Report 0-4538-1. Cooperative Research Program, Texas Transportation Institute, Texas A&M University System, College Station, 2004.
5. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets,* 5th ed. Washington, D.C., 2004.

Some of these references can be found in the Technical Reference Library in Volume 4.

CHAPTER 23
OFF-STREET PEDESTRIAN AND BICYCLE FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

Off-street pedestrian and bicycle facilities are facilities that (*a*) are used only by nonmotorized modes and (*b*) are not considered part of an urban street or transit facility. The second part of this definition excludes facilities located directly along an urban street (e.g., bicycle lanes or sidewalks). In general, off-street facilities include those for which the characteristics of motor vehicle traffic do not play a strong role in determining the quality of service from the perspective of bicyclists and pedestrians. Thus, a shared-use path only 10 ft from a roadway but separated by a sound barrier may be considered an off-street facility, whereas a sidepath with a 10-ft planted buffer separating it from the roadway would generally be considered an on-street facility.

Pedestrian capacity concepts are the same across facility types (i.e., exclusive off-street facilities, on-street facilities, and transit facilities). However, LOS thresholds for transit facilities allow higher levels of crowding for a given LOS than do the thresholds for nontransit facilities.

Off-street facilities are those generally located more than 35 ft from a roadway, although the exact distance may vary on the basis of the local context.

In general, facilities located within approximately 35 ft of an urban street are not considered off-street, although the precise definition of "off-street" varies by facility as described earlier. These types of pedestrian and bicycle facilities are covered in Chapter 16, Urban Street Facilities, and Chapter 17, Urban Street Segments. The definition also excludes crosswalks and queuing areas; these are addressed in each of the chapters on intersections (Chapters 18–21). Pedestrian components of transit facilities are addressed in the *Transit Capacity and Quality of Service Manual* (*1*). The 35-ft threshold is based on studies of pedestrian and bicycle facilities (*2–4*) in which it was found that motor vehicle traffic influenced pedestrian and bicycle quality of service on facilities located within at least this distance of the roadway.

Chapter 23, Off-Street Pedestrian and Bicycle Facilities, provides capacity and level-of-service (LOS) estimation procedures for the types of facilities shown below. Examples of each of the following facility types can be found in Chapter 3, Modal Characteristics:

- Walkways: paved paths, ramps, and plazas that are generally located more than 35 ft from an urban street as well as streets reserved for pedestrian traffic on a full- or part-time basis;
- Stairways: staircases that are part of a longer pedestrian facility;
- Shared-use paths: paths physically separated from highway traffic for the use of pedestrians, bicyclists, runners, inline skaters, and other users of nonmotorized modes; and
- Exclusive off-street bicycle paths: paths physically separated from highway traffic for the exclusive use of bicycles.

ANALYSIS BOUNDARIES

The analysis of off-street pedestrian and bicycle facilities occurs at the segment level. A segment ends and a new segment begins when any of the following occur:

- There is a street crossing;
- The width of the facility changes significantly;
- There is an intersection with another exclusive pedestrian or bicycle facility, where user volumes change significantly or cross flows are created; or
- The type of facility changes (e.g., where a walkway becomes a stairway).

LOS CRITERIA

The LOS thresholds defined for each of the off-street pedestrian and bicycle facilities are presented in this section. Three types of service measures are defined:

- For pedestrians on exclusive pedestrian facilities, *pedestrian space* (square feet per pedestrian);
- For pedestrians on facilities shared by pedestrians and bicycles, the number of *bicycle meeting and passing events per hour*; and
- For bicycles on both shared-use and exclusive paths, a *bicycle LOS score* incorporating meetings per minute, active passings per minute, presence of a centerline, path width, and delayed passings.

Exhibit 23-1 through Exhibit 23-5 provide five LOS tables: four for pedestrian facilities and one for bicycle facilities. As described in Chapter 4, Traffic Flow and Capacity Concepts, pedestrian flow rates and speeds are directly related to the average space occupied by a pedestrian. These values are given for reference in the space-based LOS tables along with the corresponding range of volume-to-capacity (v/c) ratios; however, the actual LOS in those tables are based on space per pedestrian.

LOS does not reflect whether a facility complies with the ADA or other standards.

The LOS thresholds are based on user perception research where available and in other cases on expert judgment. LOS does not reflect whether a facility complies with the Americans with Disabilities Act (ADA) or other standards.

Walkways

The walkway LOS tables apply to paved pedestrian paths, pedestrian zones (exclusive pedestrian streets), walkways and ramps with up to a 5% grade, and pedestrian walking zones through plaza areas. Exhibit 23-1 applies when pedestrian flow along the facility is random. Exhibit 23-2 applies when platoons of pedestrians form along the facility, for example, when a signalized crosswalk is located at one end of the portion of the facility being analyzed.

Cross flows occur at the intersection of two approximately perpendicular pedestrian streams (e.g., where two walkways intersect or at a building entrance). Because of the increased number of conflicts that occur between pedestrians, walkway capacity is lower in a cross-flow situation than along other

parts of the walkway. In cross-flow locations, the LOS E–F threshold is 13 ft^2/p, as indicated in the notes for Exhibit 23-1 and Exhibit 23-2.

Exhibit 23-1
Average Flow LOS Criteria for Walkways

LOS	Average Space (ft^2/p)	Related Measures: Flow Rate (p/min/ft)[a]	Average Speed (ft/s)	*v/c* Ratio[b]	Comments
A	>60	≤5	>4.25	≤0.21	Ability to move in desired path, no need to alter movements
B	>40–60	>5–7	>4.17–4.25	>0.21–0.31	Occasional need to adjust path to avoid conflicts
C	>24–40	>7–10	>4.00–4.17	>0.31–0.44	Frequent need to adjust path to avoid conflicts
D	>15–24	>10–15	>3.75–4.00	>0.44–0.65	Speed and ability to pass slower pedestrians restricted
E	>8–15[c]	>15–23	>2.50–3.75	>0.65–1.00	Speed restricted, very limited ability to pass slower pedestrians
F	≤8[c]	Variable	≤2.50	Variable	Speeds severely restricted, frequent contact with other users

Notes: Exhibit 23-1 does not apply to walkways with steep grades (>5%). See the Special Cases section for further discussion.
[a] Pedestrians per minute per foot of walkway width.
[b] *v/c* ratio = flow rate/23. LOS is based on average space per pedestrian.
[c] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Exhibit 23-2
Platoon-Adjusted LOS Criteria for Walkways

LOS	Average Space (ft^2/p)	Related Measure: Flow Rate[a] (p/min/ft)[b]	Comments
A	>530	≤0.5	Ability to move in desired path, no need to alter movements
B	>90–530	>0.5–3	Occasional need to adjust path to avoid conflicts
C	>40–90	>3–6	Frequent need to adjust path to avoid conflicts
D	>23–40	>6–11	Speed and ability to pass slower pedestrians restricted
E	>11–23[c]	>11–18	Speed restricted, very limited ability to pass slower pedestrians
F	≤11[c]	>18	Speeds severely restricted, frequent contact with other users

Notes: [a] Rates in the table represent average flow rates over a 5-min period. Flow rate is directly related to space; however, LOS is based on average space per pedestrian.
[b] Pedestrians per minute per foot of walkway width.
[c] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Stairways

Exhibit 23-3 provides the LOS criteria for stairways.

Exhibit 23-3
LOS Criteria for Stairways

LOS	Average Space (ft^2/p)	Related Measures: Flow Rate (p/min/ft)[a]	*v/c* Ratio[b]	Comments
A	>20	≤5	≤ 0.33	No need to alter movements
B	>17–20	>5–6	>0.33–0.41	Occasional need to adjust path to avoid conflicts
C	>12–17	>6–8	>0.41–0.53	Frequent need to adjust path to avoid conflicts
D	>8–12	>8–11	>0.53–0.73	Limited ability to pass slower pedestrians
E	>5–8	>11–15	>0.73–1.00	Very limited ability to pass slower pedestrians
F	≤5	Variable	Variable	Speeds severely restricted, frequent contact with other users

Notes: [a] Pedestrians per minute per foot of walkway width.
[b] *v/c* ratio = flow rate/15. LOS is based on average space per pedestrian.

Pedestrians on Shared-Use Paths

Exhibit 23-4 shows LOS criteria for paths shared between pedestrians and bicycles.

Exhibit 23-4
Pedestrian LOS Criteria for Shared-Use Paths

LOS	Weighted Event Rate/h	Related Measure Bicycle Service Flow Rate per Direction (bicycles/h)	Comments
A	≤38	≤28	Optimum conditions, conflicts with bicycles rare
B	>38–60	>28–44	Good conditions, few conflicts with bicycles
C	>60–103	>44–75	Difficult to walk two abreast
D	>103–144	>75–105	Frequent conflicts with cyclists
E	>144–180	>105–131	Conflicts with cyclists frequent and disruptive
F	>180	>131	Significant user conflicts, diminished experience

Notes: An "event" is a bicycle meeting or passing a pedestrian.
Bicycle service volumes are shown for reference and are based on a 50/50 directional split of bicycles; LOS is based on number of events per hour and applies to any directional split.

Exclusive and Shared Bicycle Facilities

Exhibit 23-5 provides LOS criteria for bicyclists on both shared-use and exclusive off-street paths.

Exhibit 23-5
LOS Criteria for Bicycles on Shared-Use and Exclusive Paths

LOS	Bicycle LOS Score	Comments
A	>4.0	Optimum conditions, ample ability to absorb more riders
B	>3.5–4.0	Good conditions, some ability to absorb more riders
C	>3.0–3.5	Meets current demand, marginal ability to absorb more riders
D	>2.5–3.0	Many conflicts, some reduction in bicycle travel speed
E	>2.0–2.5	Very crowded, with significantly reduced bicycle travel speed
F	≤2.0	Significant user conflicts and diminished experience

REQUIRED INPUT DATA

The input data required to perform an analysis differ depending on the type of facility and user being analyzed. Exhibit 23-6 shows the required input data for each of the facility types addressed in this chapter.

Exhibit 23-6
Required Input Data by Exclusive Pedestrian and Bicycle Facility Type

Pedestrian/Bicycle Facility	Required Input Data	Symbol
Walkways, stairways	Effective walkway width	E_W
	Peak 15-min pedestrian volume	v_{15}
Pedestrians on shared-use paths	Directional hourly bicycle volumes	Q_b
	Mean pedestrian speed	S_p
	Mean bicycle speed	S_b
Bicycles on shared-use and exclusive paths	Directional hourly path volumes	Q_T
	Path mode split by user group	p_i
	Path peak hour factor	*PHF*
	Mean and standard deviation of speed by user group	μ_i, σ_i
	Path width	--
	Presence of centerline on path	*CL*
	Proportion of users blocking two lanes by user group (three- and four-lane paths only)	P_b

SCOPE OF THE METHODOLOGY

Methodologies are described for evaluating the LOS of pedestrian and bicycle facilities that are separate from, and unaffected by, motor vehicle traffic. Other chapters in Volumes 2 and 3 provide methodologies for determining pedestrian and bicycle LOS on roadway system elements with motor vehicle traffic. The procedures may be applied in an approximate manner to pedestrian

zones (exclusive pedestrian streets), plazas, and ramps with grades exceeding 5%, as described later in the Special Cases section.

The analysis methodologies are based solely on facility characteristics and do not consider external factors that may also affect quality of service, such as weather, landscaping, adjacent land uses, and lighting conditions, which may also affect users' perceptions of a facility.

Much of the material in this chapter is the result of research sponsored by the Federal Highway Administration (*5–7*). Both commuter and recreational bicyclists were included in the off-street bicycle path research (*7*).

LIMITATIONS OF THE METHODOLOGY

In this chapter each of the facilities is treated from the point of view of pedestrians or bicyclists. Procedures for assessing the impact of pedestrians and bicyclists on other facility users (e.g., inline skaters) are not considered. Additional information on other users may be found elsewhere (*8*). The methodology does not address LOS for pedestrians with disabilities, including vision or mobility impairments. The reader is encouraged to consult material published by the United States Access Board to ensure compliance with the ADA.

The methodology does not address the impact of roadway crossings on the LOS of off-street paths.

The analysis methodologies presented here do not consider the continuity of walkways, bikeways, and shared-use paths in determining the LOS. Facilities that are interrupted with frequent roadway crossings will provide lower capacities and travel speeds than facilities with long, uninterrupted stretches. In addition, roadway crossings, especially crossings of high-volume or high-speed facilities, may negatively affect the pedestrian and bicycle environment and user perceptions of quality of service. However, the methodologies described here only consider discrete, uninterrupted facilities and do not assess the impact of intersections with other facilities.

Pedestrian Facilities

The capacity of pedestrian facilities is based on research conducted on constrained facilities (e.g., bridges and underground passageways), where there is no opportunity for pedestrians to walk outside the designated area. Off-street pedestrian facilities, in contrast, typically have no barriers keeping pedestrians to the designated path. As a result, these facilities reach effective failure (i.e., pedestrian spillover) at densities less than their capacity. For this reason, in combination with considerations of general pedestrian comfort, off-street walkways are desirably designed to achieve LOS C or better, based on pedestrian space, rather than for capacity conditions. The methodologies are generally appropriate regardless of the type of surface used for the pedestrian facility.

Where the opportunity exists, pedestrians will spill over the edges of a walkway at densities below capacity.

Exclusive Bicycle Facilities

The methodology for exclusive bicycle facilities is based on research conducted only on paved surfaces and may not be applicable to soft surfaces such as gravel, dirt, or wood chips.

The exclusive bicycle facility methodology may not be applicable to facilities with soft surfaces.

The pedestrian shared-use-path methodology does not account for the effects of nonbicyclist users of the path on pedestrian LOS.

Shared-Use Paths

The methodology for shared-use paths does not account for the effect on pedestrian LOS of path width or the impact of meeting and passing events. No credible data were found on fixed objects and their effects on users of these types of facilities. The methodology also does not account for the effect of nonbicyclist users of the path (e.g., skateboarders, inline skaters) on pedestrians. However, it is expected that pedestrians will often encounter these users on shared-use paths and that because of their higher speeds, these users can have a negative effect on pedestrian LOS.

The methodology for bicycle LOS on shared-use paths incorporates the effects of five user groups: bicyclists, pedestrians, runners, inline skaters, and child bicyclists. However, several path user groups that may be a part of the mix on some trails are not incorporated, including push scooter users, wheelchair users, equestrians, cross-country skiers, and users of electric vehicles. The methodology is based on research conducted only on paved surfaces and may not be applicable to soft surfaces such as gravel, dirt, or wood chips. The methodology is not applicable for paths wider than 20 ft. This methodology was developed from data collected on two-way paths but may be applied to one-way paths by setting opposing volumes equal to zero.

Some shared-use paths are signed or striped, or both, to segregate pedestrian and bicycle traffic. The research that developed the shared-use-path methodology did not address those kinds of paths; guidance on such paths may be found in the Special Cases section.

2. METHODOLOGY

OVERVIEW

Sidewalks and bicycle facilities along urban streets are addressed in Chapter 17, Urban Street Segments.

Bicycle facilities on multilane and two-lane highways are addressed in Chapters 14 and 15, respectively.

Off-street pedestrian and bicycle facilities serve only nonmotorized traffic and are separated from motor vehicle traffic to the extent that such traffic does not affect their quality of service. Thus, although sidewalks primarily serve only pedestrians, they are not addressed in this chapter—the quality of service afforded to pedestrians on sidewalks depends in part on the presence and characteristics of the adjacent motor vehicle traffic.

Procedures for estimating LOS are separated into three main categories: exclusive pedestrian facilities, exclusive bicycle facilities, and shared pedestrian and bicycle facilities. Separate methodologies are provided to assess pedestrian and bicycle LOS on shared facilities.

There are three general categories of exclusive pedestrian facilities: walkways, cross-flow areas, and stairways. The LOS thresholds for each category are different, but all are based on the concept of space per pedestrian, which is a measure of pedestrian comfort and mobility. Exhibit 23-7 illustrates the steps taken to determine the LOS of exclusive off-street pedestrian facilities.

Exhibit 23-7
Flowchart for Analysis of Exclusive Off-Street Pedestrian Facilities

Step 1: Determine the effective walkway width

↓

Step 2: Calculate the pedestrian flow rate

↓

Step 3: Calculate the average pedestrian space

↓

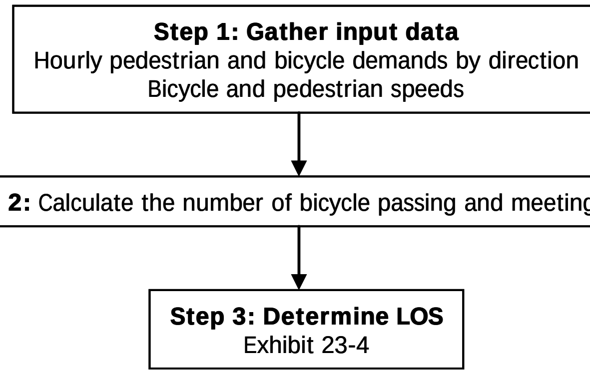
Step 4: Determine LOS
Exhibit 23-1 (walkways without platooning)
Exhibit 23-2 (walkways with platooning)
Exhibit 23-3 (stairways)

↓

Step 5: Calculate the volume-to-capacity ratio

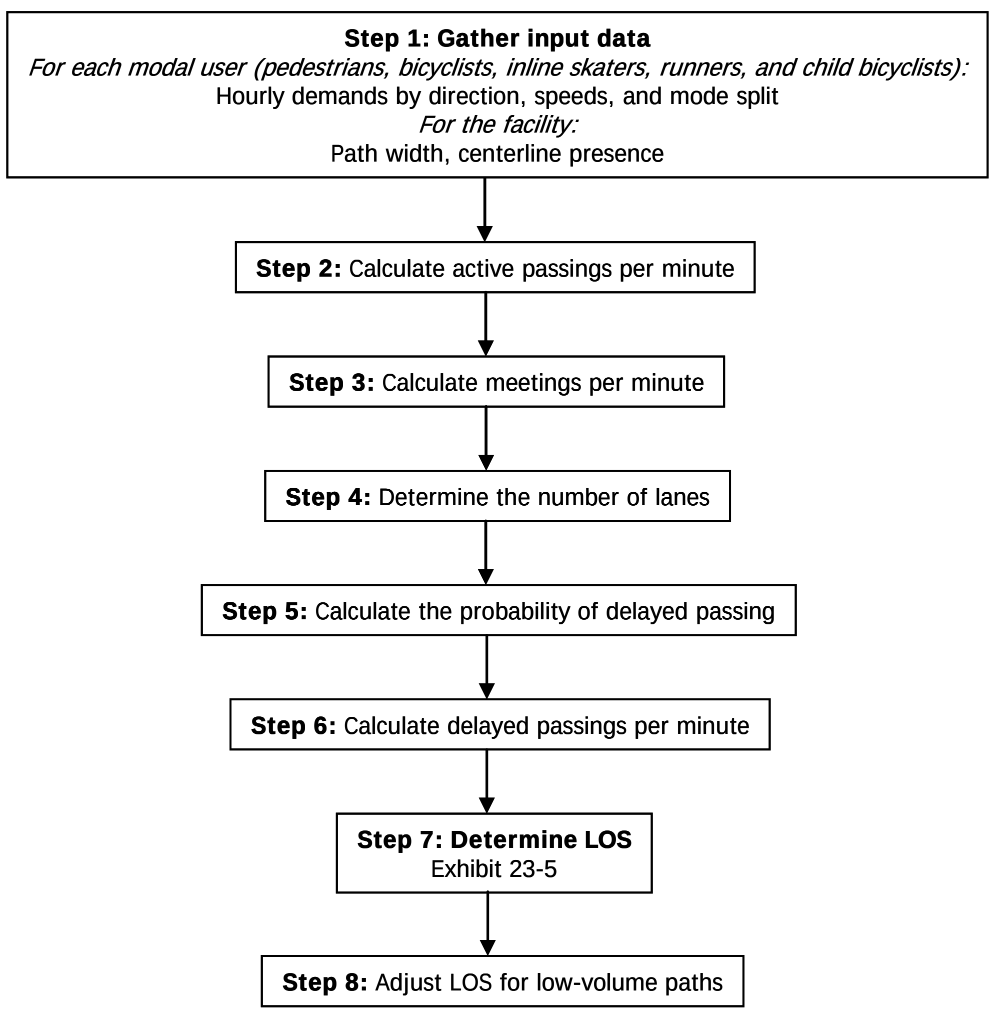
LOS for pedestrians on shared off-street bicycle and pedestrian facilities is based on the number of events during which a pedestrian either meets an oncoming bicyclist or is passed by a bicyclist. As the number of events increases, the pedestrian LOS decreases because of reduced comfort. Exhibit 23-8 shows the steps taken to determine shared-facility LOS.

Exhibit 23-8
Flowchart for Analysis of Pedestrian LOS on Shared Off-Street Facilities



Bicycle LOS on exclusive and shared-use off-street bicycle facilities is based on user perceptions of how the LOS of shared-use paths changes according to several different factors. These factors are combined into a single bicycle LOS score. LOS thresholds relate to a specific range of LOS score values. Exhibit 23-9 shows the steps taken to determine the LOS of off-street bicycle facilities.

Exhibit 23-9
Flowchart for Analysis of Bicycle LOS on Off-Street Facilities



EXCLUSIVE OFF-STREET PEDESTRIAN FACILITIES

Step 1: Determine Effective Walkway Width

Walkways and Cross-Flow Areas

Effective walkway width is the portion of a walkway that can be used effectively by pedestrians. Various types of obstructions and linear features, discussed below, reduce the walkway area that can be effectively used by pedestrians. The effective walkway width at a given point along the walkway is computed as follows:

$$W_E = W_T - W_O$$

Equation 23-1

where

W_E = effective walkway width (ft),

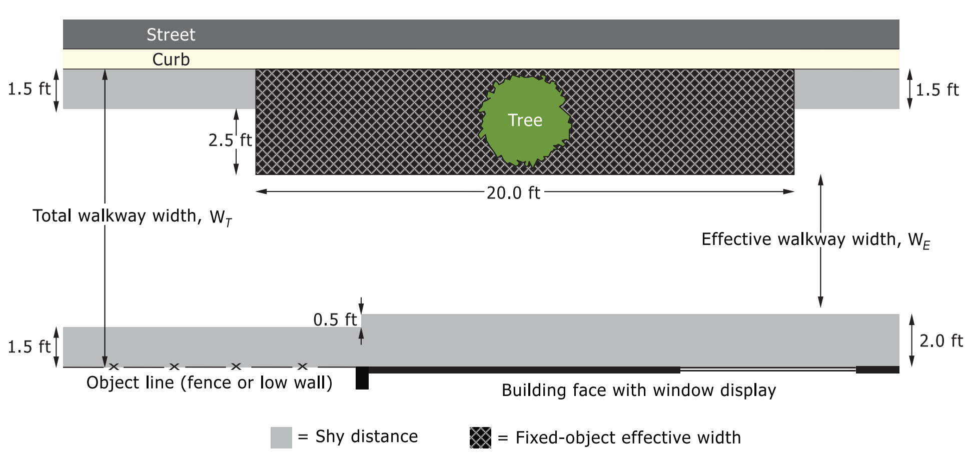
W_T = total walkway width at a given point along walkway (ft), and

W_O = sum of fixed-object effective widths and linear-feature shy distances at a given point along walkway (ft).

Shy distance is a buffer that pedestrians leave between themselves and linear objects along a walkway, such as curbs and building faces.

Exhibit 23-10 illustrates a portion of a sidewalk or walkway. The general concepts shown are applicable both to sidewalks along urban streets and to exclusive off-street paths not located adjacent to a street. Linear features such as the street curb, the low wall, and the building face each have associated shy distances. The *shy distance* is the buffer that pedestrians give themselves to avoid accidentally stepping off the curb, brushing against a building face, or getting too close to other pedestrians standing under awnings or window shopping. Fixed objects, such as the tree, have effective widths associated with them. The *fixed-object effective width* includes the object's physical width, any functionally unusable space (e.g., the space between a parking meter and the curb or the space in front of a bench occupied by people's legs and belongings), and the buffer given the object by pedestrians.

Exhibit 23-10
Width Adjustments for Fixed Obstacles



The concept of effective width applies to both on-street and off-street facilities. Because of the proximity of the street in Exhibit 23-10, the sidewalk here would be considered an on-street pedestrian facility.

The street is shown so that all factors that can influence the effective width of walkways can be depicted in one place.

Exhibit 23-10 also shows that the effective width of a fixed object extends over an effective length that is considerably longer than the object's physical length. The effective length represents the portion of the walkway that is functionally unusable because pedestrians need to move to one side ahead of

time to get around a fixed object. The effective length of a fixed object is assumed to be five times the object's effective width.

Typically, a walkway operational analysis evaluates the portion of the walkway with the narrowest effective width, since this section forms the constraint on pedestrian flow. A design analysis identifies the minimum effective width that must be maintained along the length of the walkway to avoid pedestrian queuing or spillover.

Exhibit 23-11 gives the effective widths of a variety of typical fixed objects found along on- and off-street pedestrian facilities. The values in Exhibit 23-11 can be used when specific walkway configurations are not available.

Exhibit 23-11
Typical Fixed-Object Effective Widths

Fixed Object	Effective Width (ft)
Street Furniture	
Light pole	2.5–3.5
Traffic signal poles and boxes	3.0–4.0
Fire alarm boxes	2.5–3.5
Fire hydrants	2.5–3.0
Traffic signs	2.0–2.5
Parking meters	2.0
Mail boxes (1.7 ft x 1.7 ft)	3.2–3.7
Telephone booths (2.7 ft x 2.7 ft)	4.0
Trash cans (1.8 ft diameter)	3.0
Benches	5.0
Bus shelters (on sidewalk)	6.0–7.0
Public Underground Access	
Subway stairs	5.5–7.0
Subway ventilation gratings (raised)	6.0+
Transformer vault ventilation gratings (raised)	6.0+
Landscaping	
Trees	3.0–4.0
Planter boxes	5.0
Commercial Uses	
Newsstands	4.0–13.0
Vending stands	Variable
Advertising and store displays	Variable
Sidewalk cafés (two rows of tables)	7.0
Building Protrusions	
Columns	2.5–3.0
Stoops	2.0–6.0
Cellar doors	5.0–7.0
Standpipe connections	1.0
Awning poles	2.5
Truck docks (trucks protruding)	Variable
Garage entrance/exit	Variable
Driveways	Variable

Source: Pushkarev and Zupan (*9*).

See Exhibit 23-10 for shy distances associated with curbs and building faces.

Stairways

Pedestrians tend to walk in lines or lanes on stairways; thus, meaningful increases in capacity are related to the number of pedestrian lanes available.

A stairway's capacity is largely affected by its width. Unlike walking on a level surface, traversing stairs tends to make people walk in lines or lanes. The width of a stairway determines both the number of distinct lines that can traverse the stair and the side-to-side spacing between them, affecting both the ability of faster pedestrians to pass slower-moving pedestrians and the level of interference between adjacent lines of people. The consequence is that meaningful increases in capacity are not linearly proportional to the width but occur in increments of about 30 in. (*1*).

On stairways (in contrast to walkways), a minor pedestrian flow in the opposing direction can result in reduced capacity disproportionate to the magnitude of the reverse flow. As a result, a small reverse flow should be assumed to occupy one pedestrian lane or 30 in. of the stair's width. For a stairway with an effective width of 60 in. (5 ft), a small reverse flow could consume half its capacity (*1*). The allowance for small reverse flows, when used, is included as part of the W_O term in Equation 23-1.

Small reverse flows on stairways should be assumed to use one pedestrian lane (30 in.) of width.

Step 2: Calculate Pedestrian Flow Rate

Walkways and Cross-Flow Areas

Hourly pedestrian demands is used as an input to the analysis. Consistent with the general analysis procedures used throughout the HCM, hourly demand is usually converted into peak 15-min flows, so that LOS is based on the busiest 15 consecutive minutes during an hour:

$$v_{15} = \frac{v_h}{4 \times PHF}$$

Equation 23-2

where

v_{15} = pedestrian flow rate during peak 15 min (p/h),

v_h = pedestrian demand during analysis hour (p/h), and

PHF = peak hour factor.

However, if peak-15-min pedestrian volumes are available, the highest 15-min volume can be used directly without the application of a peak hour factor.

Next, the peak 15-min flow is converted into a unit flow rate (pedestrians per minute per foot of effective path width):

$$v_p = \frac{v_{15}}{15 \times W_E}$$

Equation 23-3

where v_p is pedestrian flow per unit width (p/ft/min) and all other variables are as previously defined.

Stairways

Critical pedestrian flows on stairs occur in the up direction.

Because pedestrians use more energy to ascend stairs than to descend them, lower flow rates typically occur in the ascending direction. For this reason, when stairs serve both directions simultaneously or when the same stairway will be used primarily in the up direction during some time periods and primarily in the down direction during other time periods, the upward flow rate should be used for analysis and design (*1*). The calculation of pedestrian flow rate for stairways is otherwise the same as that described for walkways and cross-flow areas.

$Space = \frac{1}{Density}$

Step 3: Calculate Average Pedestrian Space

The service measure for walkways is *pedestrian space,* the inverse of density. Pedestrian space can be directly observed in the field by measuring a sample area of the facility and determining the maximum number of pedestrians at a given time in that area. The pedestrian unit flow rate is related to pedestrian space and speed:

Equation 23-4

$$A_p = \frac{S_p}{v_p}$$

where

A_p = pedestrian space (ft^2/p),

S_p = pedestrian speed (ft/min), and

v_p = pedestrian flow per unit width (p/ft/min).

Step 4: Determine LOS

Walkways with Random Pedestrian Flow

Where pedestrian flow on the path is not influenced by platooning (see next subsection), Exhibit 23-1 should be used to determine pedestrian LOS.

Ramps with grades of 5% or less can be treated as walkways for the purpose of determining LOS.

Research (*9–11*) has shown that pedestrian speeds on ramps with grades up to 5% are not significantly different from speeds on level walkways but that speeds decrease at higher grades. Therefore, the walkway LOS values are also applicable to ramps with grades of 5% or less. Ramps with steeper grades are discussed later in this chapter in the Special Cases section. The walkway LOS values can also be adapted to pedestrian plazas and pedestrian zones (exclusive pedestrian streets), as discussed in the Special Cases section.

Walkways with Platoon Flow

Platooning on walkways.

It is important for the analyst to determine whether platooning alters the underlying assumptions of random flow in the LOS calculation. Platoons can arise, for example, if entry to a walkway segment is controlled by a traffic signal at a street crossing or if pedestrians arrive at intervals on transit vehicles.

Where platooning occurs, the pedestrian flow is concentrated over short time periods rather than being distributed evenly throughout the peak 15-min analysis period. The available space for the typical pedestrian under these circumstances is much more constrained than the average space available with random arrival would indicate. There is no strict definition for what constitutes platooning rather than random flow; observations of local conditions and engineering judgment should be used to determine the most relevant design criteria (i.e., platoons versus random flow).

If platooning occurs, Exhibit 23-2 should be used to determine LOS. Research (*9*) indicates that impeded flow starts at 530 ft^2/p, which is equivalent to a flow rate of 0.5 p/min/ft. This value is used as the LOS A–B threshold. The same research shows that jammed flow in platoons starts at 11 ft^2/p, which is equivalent to 18 p/min/ft. This value is used as the LOS E–F threshold.

Cross-Flow Areas

Cross-flow LOS thresholds are identical to those for walkways, except for the LOS E–F threshold.

A cross flow is a pedestrian flow that is approximately perpendicular to and crosses another pedestrian stream, for example, at the intersection of two walkways or at a building entrance. In general, the lesser of the two flows is referred to as the cross-flow condition. The same procedure used to estimate walkway space is used to analyze pedestrian facilities with cross flows. As shown in the footnotes to Exhibit 23-1 and Exhibit 23-2, the LOS E threshold (i.e., capacity) in cross-flow situations occurs at a lower density than that for walkways without cross flows (*12*).

Stairways

Research (*13*) has developed LOS thresholds based on the Institute of Transportation Engineers stairway standards, which provide the space and flow values given in Exhibit 23-3. As with walkways, stairway LOS is described by the service measure of pedestrian space, expressed as square feet per pedestrian.

Step 5: Calculate Volume-to-Capacity Ratio

The volume-to-capacity (v/c) ratio can be computed by using the following values of capacity for various exclusive pedestrian facilities:

- Walkways with random flow: 23 p/min/ft,
- Walkways with platoon flow (average over 5 min): 18 p/min/ft,
- Cross-flow areas: 17 p/min/ft (sum of both flows), and
- Stairways (up direction): 15 p/min/ft in the ascending direction.

SHARED-USE PATHS

Shared-use pedestrian–bicycle paths typically are open to users of nonmotorized modes such as bicyclists, skateboarders, and wheelchair users. Shared-use paths are often constructed to serve areas without city streets and to provide recreational opportunities for the public. These paths are also common on university campuses, where motor vehicle traffic and parking are often restricted. In the United States, there are few paths exclusively for pedestrians—most off-street paths are for shared use.

Bicycles—because of their markedly higher speeds—have a negative effect on pedestrian capacity and LOS on shared-use paths. However, it is difficult to establish a bicycle–pedestrian equivalent because the relationship between the two differs depending on their respective flows and directional splits, among other factors. This section covers pedestrian LOS on shared-use paths. Bicyclists have a different perspective, as discussed in the following section.

Step 1: Gather Input Data

The following input data are required for the analysis:

- Hourly pedestrian and bicycle demands by direction, and
- Average pedestrian and bicycle speeds.

Step 2: Calculate Number of Bicycle Passing and Meeting Events

LOS is based on the overtaking of pedestrians by bicyclists. Pedestrian-to-pedestrian interaction is typically negligible.

LOS for shared-use paths is based on hindrance. Research (*14*) has established LOS thresholds for pedestrians based on the frequency of passing (in the same direction) and of meeting (in the opposite direction) other users. Because pedestrians seldom overtake other pedestrians, pedestrian LOS on a shared-use path depends on the frequency with which the average pedestrian is met and overtaken by bicyclists (*14*). However, the analyst should observe pedestrian behavior in the field before assuming that pedestrian-to-pedestrian interaction is negligible. The shared-use-path methodology does not account for events with users other than bicyclists (e.g., inline skaters).

The average number of passing and meeting events per hour is calculated by Equation 23-5 and Equation 23-6. These equations do not account for the range of bicycle speeds encountered in practice; however, because of the limited degree of overlap between the speed distributions of bicyclists and pedestrians, the resulting difference is practically insignificant.

For one-way paths, there are no meeting events, so only F_p, the number of passing events, needs to be calculated. Paths 15 ft or more in width may effectively operate as two adjacent one-way facilities, in which case F_m may be set to zero.

Equation 23-5

$$F_p = \frac{Q_{sb}}{PHF}\left(1 - \frac{S_p}{S_b}\right)$$

Equation 23-6

$$F_m = \frac{Q_{ob}}{PHF}\left(1 + \frac{S_p}{S_b}\right)$$

where

F_p = number of passing events (events/h),

F_m = number of meeting events (events/h),

Q_{sb} = bicycle demand in same direction (bicycles/h),

Q_{ob} = bicycle demand in opposing direction (bicycles/h),

PHF = peak hour factor,

S_p = mean pedestrian speed on path (mi/h), and

S_b = mean bicycle speed on path (mi/h).

Meeting events create less hindrance than overtaking events.

Meeting events allow direct visual contact, so opposing-direction bicycles tend to cause less hindrance to pedestrians. To account for the reduced hindrance, a factor of 0.5 is applied to the meeting events on the basis of theoretical considerations (*14*). Where sufficient data are available on the relative effects of meetings and passings on hindrance, this factor can be calibrated to local conditions. Because the number of events calculated in the previous step was based on hourly demand, a peak hour factor must be applied to convert them to the equivalent demand based on peak 15-min conditions. The total number of events is

$$F = \left(F_p + 0.5F_m\right)$$

Equation 23-7

where F is the total events on the path in events per hour and the other variables are as defined previously.

Step 3: Determine LOS

Exhibit 23-4 is used to determine shared-use-path pedestrian LOS based on the total events per hour calculated in Step 2. Unlike the case for exclusive pedestrian facilities, the LOS E–F threshold does not reflect the capacity of a shared-use path but rather a point at which the number of bicycle meeting and passing events results in a severely diminished experience for the pedestrians sharing the path.

OFF-STREET BICYCLE FACILITIES

The uninterrupted-flow bicycle facility analysis is based on several factors that affect user perception.

On shared-use paths, the presence of other bicyclists and other path users can be detrimental to bicyclists by increasing bicycle delay, decreasing bicycle capacity, and reducing bicyclists' freedom of movement. Research (*7*) correlating user perceptions of comfort and enjoyment of path facilities with an objective measure of path and user characteristics serves as the basis for the LOS thresholds and methodology described in this section. The following key criteria are considered through this methodology:

- The ability of a bicyclist to maintain an optimum speed,
- The number of times that bicyclists meet or pass other path users, and
- The bicyclist's freedom to maneuver.

The results of a perception survey were used to fit a linear regression model in which the survey results served as the dependent variable. The methodology incorporates the effects of five path modes that may affect bicycle LOS: other bicyclists, pedestrians, runners, inline skaters, and child bicyclists. Five variables—meetings per minute, active passings per minute, path width, presence of a centerline, and delayed passings—are used in the model. In the special case of an exclusive off-street bicycle facility, the volume for all nonbicycle modes is assumed to be zero, and the number of passings and meetings is determined solely by the volume of bicycles.

On exclusive off-street bicycle facilities the number of passings and meetings is determined solely by the bicycle volume.

The following sections describe the steps to be taken in calculating bicycle LOS for an off-street facility.

Step 1: Gather Input Data

The methodology addresses five types of path users, or *mode groups*: bicyclists, pedestrians, runners, inline skaters, and child bicyclists. The following input data are required for each mode group:

- Hourly demand by direction in modal users per hour,
- Average mode group speed in miles per hour, and
- Proportion of all path users represented by a particular mode group (i.e., mode split).

In addition, the following data are required for the facility:

- Path width in feet, and
- Presence of a centerline stripe (yes or no).

With the hourly directional demand for the path and the path mode split, the hourly directional flow rate on the path is calculated for each of the five modes:

Equation 23-8

$$q_i = \frac{Q_T \times p_i}{PHF}$$

where

q_i = hourly directional path flow rate for user group i (modal users/h),

Q_T = total hourly directional path demand (modal users/h),

p_i = path mode split for user group i, and

PHF = peak hour factor.

Step 2: Calculate Active Passings per Minute

Active passings are defined as the number of other path users traveling in the same direction as an average bicyclist (i.e., a bicyclist traveling at the average speed of all bicycles), who are passed by that bicyclist. The average bicyclist is assumed to move at a constant speed U. The value of U should be set to the average speed of bicyclists on the facility in question; where local data are not available, the default average bicyclist speed of 12.8 mi/h may be used. The methodology for determining active passings incorporates separately the effects of each of the five mode groups described in Step 1. The speeds of path users of each mode group are assumed to be normally distributed with a mean μ_i and standard deviation σ_i^2, where i represents mode.

The average bicyclist passes only those users who (*a*) are present on the path segment when the average bicyclist enters and (*b*) exit the segment after the average bicyclist does. Thus, for a given modal user in the path when the average bicyclist enters, the probability of being passed is expressed by

Equation 23-9

$$P(v_i) = P[v_i < U(1 - \frac{x}{L})]$$

where

$P(v_i)$ = probability of passing user of mode i,

U = speed of average bicyclist (mi/h),

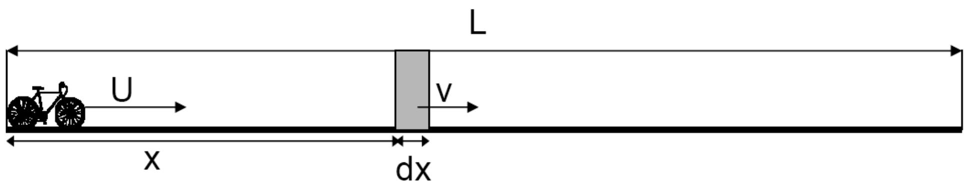
v_i = speed of path user of mode i (mi/h),

L = length of path segment (mi), and

x = distance from average bicyclist to user (mi).

Exhibit 23-12 provides a schematic of active passing events.

Exhibit 23-12
Schematic of Active Passing Events



Since v_i is distributed normally, the probability in Equation 23-9 can be calculated from the integral under the standard normal curve. By dividing the full length of the path L into n small discrete pieces each of length dx, the average probability of passing within each piece j can be estimated as the average of the probabilities at the start and end of each piece:

Equation 23-10

$$P(v_i) = 0.5[F(x - dx) + F(x)]$$

where $F(x)$ is the cumulative probability of normal distribution, and the other variables are as defined previously.

The expected number of times that the average bicyclist passes users of mode i over the entire path segment is then determined by multiplying $P(v_i)$ by the density of users of mode i and summing over all portions of the segment. The number of passings per minute is then obtained by dividing the result by the number of minutes required for the bicyclist to traverse the path segment:

Equation 23-11

$$A_i = \sum_{j=1}^{n} P(v_i) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

where

A_i = expected passings per minute of mode i by average bicyclist,

q_i = directional hourly flow rate of mode i (modal users/h),

μ_i = average speed of mode i (mi/h),

t = path segment travel time for average bicyclist (min), and

dx_j = length of discrete segment j (mi).

The other variables are as previously defined.

Research (*7*) found that setting dx equal to 0.01 mi is appropriate for the purposes of the calculations shown in Equation 23-11 and below.

Equation 23-11 provides expected passings by the average bicyclist for mode i. To determine total active passings of all modes, Equation 23-11 must be repeated for each individual mode and then summed:

Equation 23-12

$$A_T = \sum_i A_i$$

where A_T is the expected active passings per minute by the average bicyclist during the peak 15 min, and the other variables are as defined previously.

Step 3: Calculate Meetings per Minute

Meetings are defined as the number of path users traveling in the opposing direction to the average bicyclist that the average bicyclist passes on the path segment. All users present on the path when the average bicyclist enters will be

passed by the average bicyclist, assuming that no user enters or exits the path at an intermediate point:

Equation 23-13

$$M_1 = \frac{U}{60} \sum_i \frac{q_i}{\mu_i}$$

where M_1 is the meetings per minute of users already on the path segment and U is the speed of the average bicyclist in miles per hour. The other variables are as previously defined.

In addition to users already on the path segment, a number of users who have yet to enter the segment will meet the average bicyclist within the segment. The probability of this occurrence is

Equation 23-14

$$P(v_{Oi}) = P(v_i > X\frac{U}{L})$$

where

$P(v_{Oi})$ = probability of meeting opposing user of mode i,

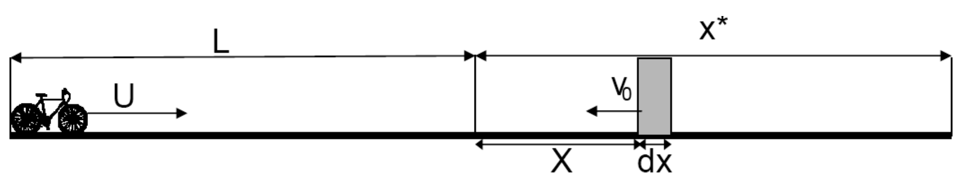
v_i = speed of path user of mode i (mi/h),

X = distance of user beyond end of path segment (mi), and

U = speed of average bicyclist (mi/h).

Since v_{Oi} is distributed normally, the probability in Equation 23-14 can be readily calculated from the area under the standard normal curve. The length of path beyond the analysis segment that may supply users who will be met by the average bicyclist is defined as x^*. By dividing x^* into n small discrete pieces, each of length dx, the average probability of meeting a modal user from each piece can be estimated by Equation 23-10. Although some meetings will occur with very fast path users located greater than L distance beyond the end of the segment when the average bicyclist enters, setting x^* equal to L is sufficient to guarantee that at least 99% of meetings will be captured (*7*). Exhibit 23-13 provides a schematic of meeting events.

Exhibit 23-13
Schematic of Meeting Events



Similar to the process for calculating number of active passings (Equation 23-11), the estimation of number of meetings with users from a particular mode group not on the path segment when the average bicyclist enters is

Equation 23-15

$$M_{2i} = \sum_{j=1}^{n} P(v_{Oi}) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

where M_{2i} is the expected meetings per minute of users of mode i located beyond the end of the path segment at the time the average bicycle enters the segment, and the other variables are as previously defined.

Finally, the total number of expected meetings per minute during the peak 15 min M_T is determined by adding M_1 to the sum of M_{2i} across all mode groups:

Equation 23-16

$$M_T = (M_1 + \sum_i M_{2i})$$

All variables are as previously defined.

In the special case of a one-way path, there are no opposing users to meet; therefore, M_T is zero.

Step 4: Determine Number of Lanes

The effective number of lanes on a shared-use path affects the number of delayed passings: as the number of lanes increases, delayed passings decrease. Even paths without painted lane markings will operate with a de facto number of lanes. The relationship between path width and the number of effective operational lanes is shown in Exhibit 23-14.

Exhibit 23-14
Effective Lanes by Path Width

Path Width (ft)	Lanes
8.0–10.5	2
11.0–14.5	3
15.0–20.0	4

Source: Hummer et al. (*7*).

Step 5: Calculate Probability of Delayed Passing

Delayed passing maneuvers occur when there is a path user ahead of the overtaking average bicyclist in the subject direction and another path user in the opposing direction, such that the average bicyclist cannot immediately make the passing maneuver. The probability of a delayed passing depends on the passing distance required, which in turn depends on both the overtaking mode and the mode of the user being passed. The passing distances that bicyclists require to pass other user modes are shown in Exhibit 23-15.

Exhibit 23-15
Required Bicycle Passing Distance

Overtaking Mode	Mode Passed	Required Passing Distance (ft)
Bicycle	Bicyclist	100
Bicycle	Pedestrian	60
Bicycle	Inline skater	100
Bicycle	Runner	70
Bicycle	Child bicyclist	70

Source: Hummer et al. (*7*).

With the values in Exhibit 23-15, the probability that a given section will be vacant of a given mode for at least the required passing distance p_i can be estimated by using a Poisson distribution. The probability of observing at least one modal user in the passing section is the complement of the probability of observing a vacant section. The probability P_{ni} of observing a blocked passing section for mode *i* is

Equation 23-17

$$P_{ni} = 1 - e^{-p_i k_i}$$

where

P_{ni} = probability of passing section's being blocked by mode *i*,

p_i = distance required to pass mode *i* (mi), and

k_i = density of users of mode *i* (users/mi).

Equation 23-17 is applicable to both the subject and opposing directions.

Two-Lane Paths

On a two-lane path, delayed passing occurs when, within the distance required to complete a pass *p*, the average bicyclist encounters one of the following: traffic in both directions, each blocking a single lane, or no traffic in the subject direction in conjunction with traffic in the opposing direction that is being overtaken by an opposing bicyclist. Note that these situations are mutually exclusive. The delayed passing probabilities in the subject and opposing directions are

Equation 23-18

$$P_{ds} = P_{no}P_{ns} + P_{no}(1 - P_{ns})(1 - P_{do})$$

Equation 23-19

$$P_{do} = P_{no}P_{ns} + P_{ns}(1 - P_{no})(1 - P_{ds})$$

where

P_{ds} = probability of delayed passing in subject direction,

P_{do} = probability of delayed passing in opposing direction,

P_{no} = probability of blocked lane in opposing direction, and

P_{ns} = probability of blocked lane in subject direction.

Solving Equation 23-18 and Equation 23-19 for P_{ds} results in

Equation 23-20

$$P_{ds} = \frac{P_{no}P_{ns} + P_{no}(1 - P_{ns})^2}{1 - P_{no}P_{ns}(1 - P_{no})(1 - P_{ns})}$$

Since P_{no} and P_{ns} are calculated from Equation 23-17, Equation 23-20 can be readily solved for P_{ds}.

Three-Lane Paths

The operations of three-lane paths are more complicated than those of two-lane paths, since there is a greater variety of possible scenarios that may occur. The methodology includes several limiting assumptions regarding user behavior:

- Bicyclists in the subject direction use only the two rightmost lanes,
- Bicyclists in the opposing direction use only the two leftmost lanes,
- Passing maneuvers occur only in the middle lane and never in the left lane, and
- Groups of users may sometimes block the two lanes allocated to that direction but cannot block all three lanes.

As a result, a delayed passing occurs in two cases: (*a*) traffic in the subject direction is blocking the rightmost lane in conjunction with opposing traffic occupying the other two lanes, or (*b*) side-by-side users are blocking the two rightmost lanes in the subject direction. The probabilities of the occurrence of a delayed passing in the subject and opposing directions are given by

$$P_{ds} = P_{ns}[P_{bo} + P_{no}(1 - P_{do})] + P_{bs}$$

Equation 23-21

$$P_{do} = P_{no}[P_{bs} + P_{ns}(1 - P_{ds})] + P_{bo}$$

Equation 23-22

where P_{bo} is the probability of two blocked lanes in the opposing direction, P_{bs} is the probability of two blocked lanes in the subject direction, and all other variables are as previously defined.

Equation 23-21 and Equation 23-22 are simultaneous equations with two unknowns, P_{ds} and P_{do}. Defining D as $P_{ds} - P_{do}$ gives the following equation:

$$D = [(P_{bs} - P_{bo}) + (P_{ns}P_{bo} - P_{no}P_{bs})]/(1 - P_{ns}P_{no})$$

Equation 23-23

Substituting Equation 23-23 into Equation 23-21 results in

$$P_{ds} = [P_{ns}(P_{bo} + P_{no}(1 + D)) + P_{bs}]/(1 + P_{ns}P_{no})$$

Equation 23-24

This model requires determining four probability parameters: specifically, P_n and P_b in each direction. Calculating these parameters requires estimating the fraction of all events in which both lanes are blocked. These parameters were established through research (*7*) in which video data of more than 4,000 path users on U.S. shared-use paths were observed. Exhibit 23-16 shows the blocking frequencies by mode.

Exhibit 23-16
Frequency of Blocking of Two Lanes

Mode	Frequency of Blocking (%)
Bicycle	5
Pedestrian	36
Inline skater	8
Runner	12
Child bicyclist	1

Source: Hummer et al. (*7*).

Therefore, P_{boi} and P_{bsi}, the probabilities that a user of mode i will block two lanes in the opposing and subject directions, respectively, are found by multiplying the frequency of blocking two lanes by a particular user of mode i (Exhibit 23-16) by the probability that a user of mode i will be encountered, which was given by Equation 23-17. This process results in

$$P_{bsi} = F_i \times P_{nsi}$$

Equation 23-25

$$P_{boi} = F_i \times P_{noi}$$

Equation 23-26

where F_i is the frequency with which mode i will block two lanes, from Exhibit 23-16, and all other variables are as previously defined. The probability that a user of any mode will block two lanes is thus given by

$$P_{bs} = \sum_i P_{bsi}$$

Equation 23-27

$$P_{bo} = \sum_i P_{boi}$$

Equation 23-28

The probabilities that only a single lane will be blocked by a user of a given mode i, P_{qsi} and P_{qoi}, are thus derived from the probability that at least one lane will be blocked (from Equation 23-17) minus the probability that two lanes will be blocked (from Equation 23-25 and Equation 23-26). These probabilities are

Equation 23-29
$$P_{nsi} = 1 - e^{-p_i k_{si}} - P_{bsi}$$

Equation 23-30
$$P_{noi} = 1 - e^{-p_i k_{oi}} - P_{boi}$$

where k_{si} and k_{oi} are the densities of users of mode i in users per mile in the subject and opposing directions, respectively, and all other variables are as previously defined.

The probabilities that a user of any mode will block a single lane are thus given by

Equation 23-31
$$P_{ns} = \sum_i P_{nsi}$$

Equation 23-32
$$P_{no} = \sum_i P_{noi}$$

The values of P_{bs} and P_{bo} from Equation 23-27 and Equation 23-28, and the values of P_{ns} and P_{no} from Equation 23-31 and Equation 23-32 can now be substituted into Equation 23-23 and Equation 23-24 to determine the probability of delayed passing, P_{ds}. This delayed passing factor was calibrated by using peak hour volumes, rather than peak 15-min volumes. Therefore, a peak hour factor is applied to convert A_T from peak 15-min flow rate conditions back to hourly conditions.

Four-Lane Paths

On four-lane paths, the methodology assumes that the path operates similarly to a divided four-lane highway, such that the probability of delayed passing is independent of opposing users, since no passing occurs in the leftmost lanes. Thus, the probability of delayed passing P_{ds} is equivalent to the probability that both subject lanes will be blocked P_{bs}, which can be found by using Equation 23-25 and Equation 23-27.

Step 6: Calculate Delayed Passings per Minute

The probability of delayed passing P_{ds}, described earlier, applies only to a single pair of modal path users (e.g., a bicyclist passing a pedestrian and opposed by a runner). The total probability of delayed passing P_{Tds} must be calculated from all modal pairs. Since there are five modes, there are five times five (25) total modal pairs that require calculation. The total probability of delayed passing is found by using

Equation 23-33
$$P_{Tds} = 1 - \prod_m (1 - P_{mds})$$

where P_{Tds} is the total probability of delayed passing and P_{mds} is the probability of delayed passing for mode m. The operator Π in Equation 23-33 indicates the product of a series of variables.

Finally, delayed passings per minute are simply the active passings per minute A_T multiplied by the total probability of delayed passing P_{Tds}:

Equation 23-34
$$\text{Delayed Passings per Minute} = A_T \times P_{Tds} \times PHF$$

This delayed passing factor was calibrated by using peak hour volumes rather than peak 15-min volumes. Therefore, a peak hour factor is applied to convert A_T from peak 15-min flow rate conditions back to hourly conditions.

Step 7: Determine LOS

The bicycle LOS score (Equation 23-35) uses inputs from Steps 2, 3, and 6 plus facility data gathered in Step 1. The equation was developed from a regression model of user responses to video clips depicting a variety of off-street bicycle facilities (*7*). The LOS C–D threshold represents the midpoint of the response scale used in the survey.

Equation 23-35

$$\text{Bicycle LOS Score} = 5.446 - 0.00809(E) - 15.86(RW) - 0.287(CL) - 0.5(DP)$$

where

E = weighted events per minute = meetings per minute + 10 × (active passings per minute);

RW = reciprocal of path width = 1/path width (ft);

CL = 1 if trail has centerline, 0 if no centerline; and

DP = min [delayed passings per minute, 1.5].

With the exception of the special cases discussed in Step 8 below, the bicyclist perception index is used directly with Exhibit 23-5 to determine bicyclist LOS on off-street facilities. As was the case with shared pedestrian facilities, the LOS E–F threshold does not reflect the capacity of an off-street bicycle facility, but rather a point at which the number of meeting and passing events results in a severely diminished experience for bicyclists using the path.

Step 8: Adjust LOS for Low-Volume Paths

For narrow paths (i.e., 8 ft in width), it is not possible to achieve LOS A or B by using Equation 23-35. Since paths with very low volumes would be expected to result in a high perceived quality of service, the following adjustments are made to the LOS results:

- All paths with five or fewer weighted events per minute are assigned LOS A.
- All paths with >5 to 10 weighted events per minute are assigned LOS B, unless Equation 23-35 would result in LOS A.

3. APPLICATIONS

This chapter's methodologies evaluate the LOS of exclusive pedestrian and bicycle facilities. The analyst must address two fundamental issues. First, the primary outputs must be identified. These may include LOS, effective width W_E, or achievable path flow rate Q_T. Second, any necessary default or estimated values must be identified. There are three basic sources of input data:

1. Default values provided in the HCM;
2. Estimates or locally derived default values developed by the user; and
3. Values derived from field measurements and observation.

When it is possible to obtain them, field measurements are preferable to default values.

DEFAULT VALUES

For pedestrians on off-street paths, a default average speed of 3.4 mi/h for pedestrians and 12.8 mi/h for bicycles can be applied in the absence of local data (*7*). Default values for off-street bicycle facilities are summarized in Exhibit 23-17:

Exhibit 23-17
Default Values for Exclusive Off-Street Bicycle Facilities

Variable	User Group	Default Value
Mode split	Bicycle	55%
	Pedestrian	20%
	Runner	10%
	Inline skater	10%
	Child bicyclist	5%
Mean speed by mode	Bicycle	12.8 mi/h
	Pedestrian	3.4 mi/h
	Runner	6.5 mi/h
	Inline skater	10.1 mi/h
	Child bicyclist	7.9 mi/h
Standard deviation of speed by mode	Bicycle	3.4 mi/h
	Pedestrian	0.6 mi/h
	Runner	1.2 mi/h
	Inline skater	2.7 mi/h
	Child bicyclist	1.9 mi/h
Proportion of users blocking two lanes by mode	Bicycle	5%
	Pedestrian	36%
	Runner	12%
	Inline skater	8%
	Child bicyclist	1%
Peak hour factor	N/A	0.85

Source: Hummer et al. (*7*).

ANALYSIS BOUNDARIES

As stated in this chapter's introduction, exclusive pedestrian and bicycle facilities are analyzed at the segment level, with segment endpoints being defined by street crossing locations, changes in path width, intersections with other paths that create cross flows or change path demand, and transition points to other types of facilities (e.g., from path to ramp). In most cases, the minimum segment length will be around 0.25 mi, and the maximum segment length will be 2 to 3 mi (*7*). Certain kinds of facilities, such as stairways, cross-flow areas, and pedestrian plazas, will have shorter segment lengths.

TYPES OF ANALYSIS

Operational Analysis

A common application of operational analysis is to compute the LOS of a facility under existing or future demand. The effective width of the facility is an input to the calculation and LOS is an output.

Design Analysis

Designing for an effective width.

Design applications require that an LOS goal be established, with the primary output being the facility design characteristics required or the maximum user volumes allowable for the LOS goal. For instance, a design analysis for a pedestrian walkway may estimate the minimum effective width W_E needed to achieve a design LOS value. In this case, the maximum pedestrian unit flow rate for the desired service level would be determined from Exhibit 23-1 or Exhibit 23-2. The effective width would be computed by solving the pedestrian unit flow-rate equation backward. To avoid pedestrian spillover, it is desirable to design a walkway to achieve LOS C or better (i.e., a maximum of 10 p/min/ft). Stairways are desirably designed to achieve LOS C or D.

Determining service volumes.

Similarly, the achievable path flow rate Q_T can be solved as the primary output. For exclusive bicycle facilities, the minimum LOS perception score for the design LOS would be determined from Exhibit 23-5. By holding all but one path user group's demand constant and solving the events equation backward, the service volume for the user group of interest can be computed.

Planning and Preliminary Engineering Analyses

Planning and preliminary engineering analyses use estimates, HCM default values, or local default values as inputs and determine LOS, bicycle flow rate, effective width, or all three, as outputs. The difference between a planning analysis and an operational or design analysis is that most or all of the input values in planning come from estimates or default values, whereas operational and design analyses tend to use field measurements or known values for most or all of the input variables.

SPECIAL CASES

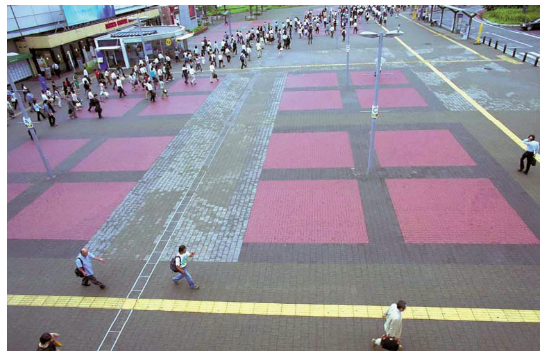
Pedestrian Plazas

Pedestrian plazas are large, paved areas that serve multiple functions, including pedestrian circulation, special events, and seating. The circulation function is of interest here, although the design of a plaza must consider how all of the functions interact. For example, queues from areas designated for food vendors may intrude into a pedestrian circulation route, reducing the route's effective width, or two circulation routes may intersect each other, creating a cross-flow area. In addition, research has shown that the circulation and amenity functions of a plaza sometimes conflict, since people tend to linger longer in plazas that do not act as thoroughfares (*9*).

The exclusive pedestrian walkway methodology can be used to analyze pedestrian circulation routes through pedestrian plazas. The methodology does

not address the need or desire to have space for amenities within a pedestrian plaza. The effective width of such a route is not as easily identified as that of a walkway, because the edges of the circulation area are often undefined. However, pedestrians will tend to take the shortest available route across the plaza, as illustrated in Exhibit 23-18.

Exhibit 23-18
Pedestrian Circulation Space in a Pedestrian Plaza



The effective width of a circulation route is influenced by the widths of the entrance and exit points to the plaza and by the presence of obstacles (e.g., walls, poles, signs, benches). Effective width may also be influenced by whether a change in texture is used to mark the transition between circulation and amenity space. Between 30% and 60% of pedestrians will use plaza space that is flush with a sidewalk, with the higher percentages applying to wider plazas and those that help cut a corner and the lower percentages applying to narrower plazas and those with obstacles (*9*).

For design applications, peak pedestrian demands through the plaza would need to be estimated. Given this information and a design LOS, a minimum effective width could be determined for each circulation route. Multiplying the width of the route by the length of the route and summing for all routes results in the space required for pedestrian circulation. Space requirements for sitting areas and other plaza functions are added to the circulation space to determine the total plaza space required.

For operational applications, an average effective width can be determined through field observation of the space occupied by pedestrians on a circulation route during peak times. Dividing an average per minute pedestrian volume by the effective width gives the pedestrian flow rate for the circulation route, from which LOS can be determined.

Pedestrian Zones

Pedestrian zones are streets dedicated to exclusive pedestrian use on a full- or part-time basis. These zones can be analyzed from an operational standpoint by using the exclusive pedestrian walkway methodology, as long as the kinds of obstructions listed in Exhibit 23-11, such as sidewalk café tables, are taken into account. Other performance measures may be considered that assess the street's attractiveness to pedestrians, since a successful pedestrian zone is expected to be relatively crowded (i.e., to have a lower LOS). Although an uncrowded zone would have a high LOS, it could be perceived by pedestrians as being a potential personal security risk, because of the lack of other users.

The HCM methodology is not suitable for pedestrian zones during times when delivery vehicles are allowed to use the street. The HCM methodology is also not applicable to the analysis of a low-speed street (e.g., a Dutch-style *woonerf*) shared by pedestrians, bicycles, and automobiles.

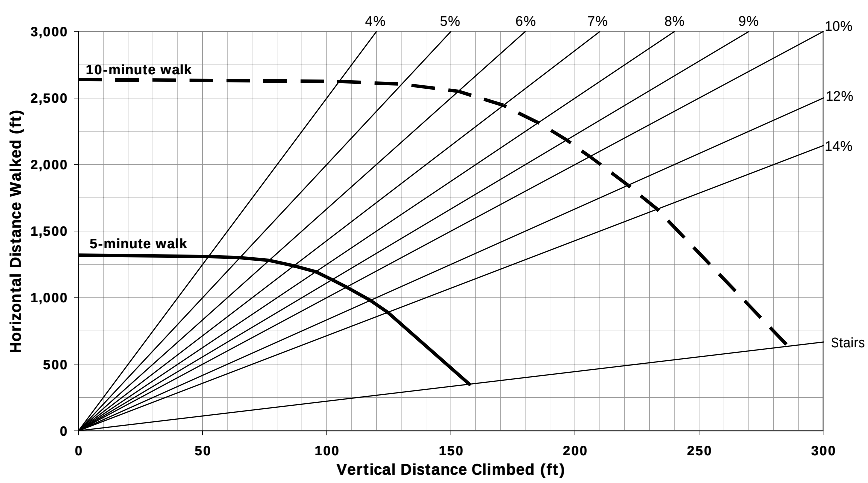
Walkways with Grades over 5%

Research (*9–11*) has shown no appreciable impact on pedestrian speed for grades up to 5%. As shown in Exhibit 23-19, above a 5% grade, walking speeds drop as grade increases, with travel on a 12% grade being about 30% slower than travel on a level surface. Grade may not have an appreciable impact on capacity, however, since the reduction in pedestrian speed is offset by closer pedestrian spacing (*9*). The stairway LOS table (Exhibit 23-3) would provide a conservative estimate of pedestrian LOS on steeper walkways.

Consult the latest version of the ADA Accessibility Guidelines for guidance on the maximum slope allowed on an accessible route.

Exhibit 23-19
Effect of Vertical Climb on Horizontal Distance Walked

LIVE GRAPH
Click here to view



Source: Municipal Planning Association (*11*).

Paths Segregating Pedestrians and Bicyclists

Some paths are signed or striped, or both, to segregate bicyclists from pedestrians. Where field observation on the path (or similar paths in the same region) indicates that path users generally comply with the regulations, up to all of the bicycle–pedestrian passing events could be converted to meeting events in proportion to the path users' compliance rate, resulting in an improved LOS. Where sufficient physical segregation of bicyclists and pedestrians occurs, it may be appropriate to treat the path as two separate facilities.

USE OF ALTERNATIVE TOOLS

To date, there is no widely used computer simulation software in the United States that is capable of describing user interactions on shared-use paths in a realistic manner. Microsimulation has been used to model pedestrian interactions on off-street pedestrian facilities. In many cases, these models were developed to model pedestrian movements within airports or transit facilities.

4. EXAMPLE PROBLEMS

Exhibit 23-20
List of Example Problems

Example Problem	Description	Application
1	Pedestrian LOS on shared-use and exclusive paths	Operational analysis
2	Bicycle LOS on a shared-use path	Planning analysis

EXAMPLE PROBLEM 1: PEDESTRIAN LOS ON SHARED-USE AND EXCLUSIVE PATHS

The Facts

The following information was collected in the field for this path:

- Q_{sb} = bicycle volume in same direction = 100 bicycles/h;
- Q_{ob} = bicycle volume in opposing direction = 100 bicycles/h;
- v_{15} = peak 15-min pedestrian volume = 100 pedestrians;
- PHF = peak hour factor = 0.83;
- S_p = average pedestrian speed = 4.0 ft/s (2.7 mi/h);
- S_b = average bicycle speed = 16.0 ft/s (10.9 mi/h); and
- No pedestrian platooning was observed.

Step 1: Gather Input Data

The shared-use-path pedestrian LOS methodology requires pedestrian and bicycle speeds and bicycle demand, all of which are available from the field measurements just given.

Step 2: Calculate Number of Bicycle Passing and Meeting Events

The number of passing events F_p is determined from Equation 23-5:

$$F_p = \frac{Q_{sb}}{PHF}\left(1 - \frac{S_p}{S_b}\right)$$

$$F_p = \frac{100 \text{ bicycles/h}}{0.83}\left(1 - \frac{4.0 \text{ ft/s}}{16.0 \text{ ft/s}}\right)$$

$$F_p = 90 \text{ events/h}$$

The number of meeting events F_m is determined from Equation 23-6:

$$F_m = \frac{Q_{ob}}{PHF}\left(1 + \frac{S_p}{S_b}\right)$$

$$F_m = \frac{100 \text{ bicycles/h}}{0.83}\left(1 + \frac{4.0 \text{ ft/s}}{16.0 \text{ ft/s}}\right)$$

$$F_m = 150 \text{ events/h}$$

The total number of events is calculated from Equation 23-7:

$$F = \left(F_p + 0.5F_m\right)$$

$$F = \left(90 \text{ events/h} + (0.5)(150 \text{ events/h})\right)$$

$$F = 165 \text{ events/h}$$

Step 3: Determine Shared-Use-Path Pedestrian LOS

The shared-use-path LOS is determined from Exhibit 23-4. The value of *F*, 165 events/h, falls into the LOS E range. Since this value is rather low, what would happen if a parallel, 5-ft-wide, pedestrian-only path were provided?

Step 4: Compare Exclusive-Path Pedestrian LOS

Step 4.1: Determine Effective Walkway Width

Assuming that no obstacles exist on or immediately adjacent to the path, the effective width would be the same as the actual width, or 5 ft. Common amenities located along pedestrian walkways include trash cans and benches. From Exhibit 23-11, these should be located at least 3.0 ft and 5.0 ft, respectively, from the edge of the path to avoid affecting the effective width.

Step 4.2: Calculate Pedestrian Flow Rate

Since a peak-15-min pedestrian volume was measured in the field, it is not necessary to use Equation 23-2 to determine v_{15}. The unit flow rate for the walkway v_p is determined from Equation 23-3 as follows:

$$v_p = \frac{v_{15}}{15 \times W_E}$$

$$v_p = \frac{100 \text{ p}}{15 \times 5 \text{ ft}}$$

$$v_p = 1.3 \text{ p/ft/min}$$

Step 4.3: Calculate Average Pedestrian Space

Average pedestrian space is determined by rearranging Equation 23-4:

$$A_p = S_p / v_p$$

$$A_p = (4.0 \text{ ft/s})(60 \text{ s/min}) / (1.3 \text{ p/ft/min})$$

$$A_p = 185 \text{ ft}^2\text{/p}$$

Step 4.4: Determine LOS

Since no pedestrian platooning was observed, Exhibit 23-1 should be used to determine the LOS. A value of 185 ft^2/min corresponds to LOS A.

Discussion

The existing mixed-use path operates at LOS E for pedestrians. Pedestrian LOS would increase to LOS A if a parallel, 5-ft-wide pedestrian path were provided.

EXAMPLE PROBLEM 2: BICYCLE LOS ON A SHARED-USE PATH

The Facts

A new shared-use path is being planned. On the basis of data from a similar facility in the region, it is estimated that the path will have a peak-hour volume of 340 users, a peak hour factor of 0.90, and a 50/50 directional split. The path will be 10 ft wide, with no obstacles, and will not have a centerline. The segment analyzed here is 3 mi long.

Step 1: Gather Input Data

Facility and overall demand data are available but not the mode split of users or the average mode group speed. Those values will need to be defaulted by using Exhibit 23-17. On the basis of the default mode split and the estimated directional split, the directional hourly volume by mode is as follows:

- Directional bicycle flow rate = 340 users/h × 0.5 × 0.55/0.90 = 104 bicycles/h;
- Directional pedestrian flow rate = 340 × 0.5 × 0.20/0.90 = 38 p/h;
- Directional runner flow rate = 340 × 0.5 × 0.10/0.90 = 19 runners/h;
- Directional inline skater flow rate = 340 × 0.5 × 0.10/0.90 = 19 skaters/h; and
- Directional child bicyclist volume = 340 × 0.5 × 0.05/0.90 = 9 child bicyclists/h.

From Exhibit 23-17, average mode group speeds and standard deviations are as follows:

- Bicycle: average speed = 12.8 mi/h, standard deviation = 3.4 mi/h;
- Pedestrian: average speed = 3.4 mi/h, standard deviation = 0.6 mi/h;
- Runner: average speed = 6.5 mi/h, standard deviation = 1.2 mi/h;
- Inline skater: average speed = 10.1 mi/h, standard deviation = 2.7 mi/h; and
- Child bicyclist: average speed = 7.9 mi/h, standard deviation = 1.9 mi/h.

Step 2: Calculate Active Passings per Minute

Active passings per minute must be calculated separately for each mode, by using Equation 23-9 through Equation 23-11. For the number of bicycles passed per minute, the path is considered as broken into n slices, each of which has a length dx of 0.01 mi, and a total path segment length L of 3 mi. Then, for the first slice, from Equation 23-9:

$$P(v_i) = P[v_i < U(1 - \frac{x}{L})]$$

$$F(x) = P[v_i < 12.8(1 - \frac{0.01}{3})] = P[v_i < 12.76] = 0.4949$$

$$F(x - dx) = P[v_i < 12.8(1 - \frac{0}{3})] = P[v_i < 12.80] = 0.5000$$

Applying Equation 23-10 and Equation 23-11 then gives the following probability of passing for the first slice:

$$P(v_i) = 0.5[F(x - dx) + F(x)]$$

$$P(v_i) = 0.5[0.5000 + 0.4949] = 0.4975$$

$$A_i = \sum_{j=1}^{n} P(v_i) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

$$A_{i1} = 0.4975 \times \frac{104}{12.8} \times \frac{1}{14}(0.01) = 0.0029$$

Repeating this procedure for all slices from 1 to n and summing the results yields

Passings of bicycles per minute = 0.0029 + A_2 + ... + A_n = 0.19

With the same methodology for each mode, the following active passings per minute are found for the other modes:

- Pedestrians, 1.74;
- Runners, 0.30;
- Inline skaters, 0.09; and
- Child bicyclists, 0.10.

Total active passings are then determined by using Equation 23-12:

$$A_T = \sum_i A_i$$

Total passings per minute = (0.19 + 1.74 + 0.30 + 0.09 + 0.10) = 2.42

Step 3: Calculate Meetings per Minute

Meetings per minute of users already on the path segment M_1 are calculated for each mode with Equation 23-13:

$$M_1 = \frac{U}{60} \sum_i \frac{q_i}{\mu_i}$$

M_1 = (12.8/60) × [(104/12.8) + (38/3.4) + (19/6.6) + (19/10.1) + (9/7.9)] = 5.36

Meetings per minute of users not yet on the path segment must be calculated separately for each mode. For the number of bicycles passed per minute, the section of path beyond the study segment is considered as broken into n slices, each of which has length dx = 0.01 mi, and a total segment length X equivalent to L (3 mi). Then, for the first slice, from Equation 23-14:

$$P(v_{Oi}) = P(v_i > X\frac{U}{L})$$

$$F(x) = P[v_i > 0.01 \times \frac{12.8}{3}] = P[v_i > 0.4267] = 0.9999$$

$$F(x) = P[v_i > 0 \times \frac{12.8}{3}] = P[v_i > 0] = 1.0000$$

Applying Equation 23-10 and Equation 23-15 then gives the probability of passing in the first slice:

$$P(v_i) = 0.5[F(x - dx) + F(x)]$$

$$P(v_i) = 0.5\ [0.99992 + 1.0000] = 0.99996$$

$$M_{2i} = \sum_{j=1}^{n} P(v_{Oi}) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

$$M_{21} = 0.99996 \times (104/12.8) \times (1/14) \times 0.01 = 0.0058$$

Repeating this procedure for all slices from 1 to n and summing the results yields

$$M_{2i} = \text{meetings of bicycles per minute} = 0.0058 + M_{22} + \ldots + M_{2n} = 1.54$$

Repeating the foregoing procedure for the other modes, the following meetings per minute are found for each mode:

- Pedestrians, 0.63;
- Runners, 0.31;
- Inline skaters, 0.31; and
- Child bicyclists, 0.16.

Total meetings are then determined by using Equation 23-16:

$$M_T = (M_1 + \sum_i M_{2i})$$

Total meetings per minute = [5.36 + 1.54 + 0.63 + 0.31 + 0.31 + 0.16] = 8.31

Step 4: Determine the Number of Lanes

From Exhibit 23-14, a 10-ft-wide path has two effective lanes.

Step 5: Calculate the Probability of Delayed Passing

From Step 4, it is clear that a path with a width of 10 ft will operate as two lanes. Therefore, delayed passings per minute must be calculated separately for each of the 25 modal pairs, by using Equation 23-17 and Equation 23-20. For instance, considering the probability of a delayed passing of a bicyclist as a result of an opposing bicyclist's overtaking a pedestrian gives the following:

$$P_{ni} = 1 - e^{-p_i k_i}$$

$$P_{ns} = 1 - \exp[-(100/5280) \times (1/0.90\ (94/12.8))] = 1 - 0.858 = 0.142$$

$$P_{no} = 1 - \exp[-(100/5280) \times (1/0.90\ (38/3.4))] = 1 - 0.810 = 0.190$$

Substituting into Equation 23-20 then yields P_{ds}:

$$P_{ds} = \frac{P_{no}P_{ns} + P_{no}(1-P_{ns})^2}{1-P_{no}P_{ns}(1-P_{no})(1-P_{ns})}$$

$$P_{ds} = \frac{0.190 \times 0.142 + 0.190(1-0.142)^2}{1-0.190 \times 0.142(1-0.190)(1-0.142)} = 0.1698$$

Performing the foregoing procedures for each of the 25 modal pairs and applying Equation 23-33 gives the total probability of delayed passing:

$$P_{Tds} = 1 - \prod_m 1 - P_{mds}$$

$$P_{Tds} = 1 - (1 - 0.1698)(1 - P_{2ds}) \ldots (1 - P_{mds}) = 0.8334$$

Thus, the probability of delayed passing is 83.34%.

Step 6: Determine Delayed Passings per Minute

Equation 23-34 is used to determine the total number of delayed passings per minute:

$$\text{Delayed passings per minute} = A_T \times P_{Tds} \times PHF$$

$$\text{Total delayed passings per minute} = 0.8334 \times 2.42 \times 0.90 = 1.82$$

The delayed passing factor has a maximum of 3. The total delayed passings per minute (1.82) is less than 3. Therefore the delayed passing factor is set to 1.82.

Step 7: Calculate LOS

Equation 23-35 is used to determine the bicycle LOS score for the path:

$$\text{Bicycle LOS Score} =$$
$$5.446 - 0.00809\,[8.58 + (10 \times 2.42)] - 15.86(1/10) - 0.287(0) - 0.5(1.82) = 2.69$$

Because the bicyclist perception index is between 2.5 and 3.0, the facility operates at LOS D according to Exhibit 23-5.

Results

The results indicate that the path would operate close to its functional capacity. A slightly wider path would provide three effective lanes and a better LOS.

5. REFERENCES

Many of these references can be found in the Technical Reference Library in Volume 4.

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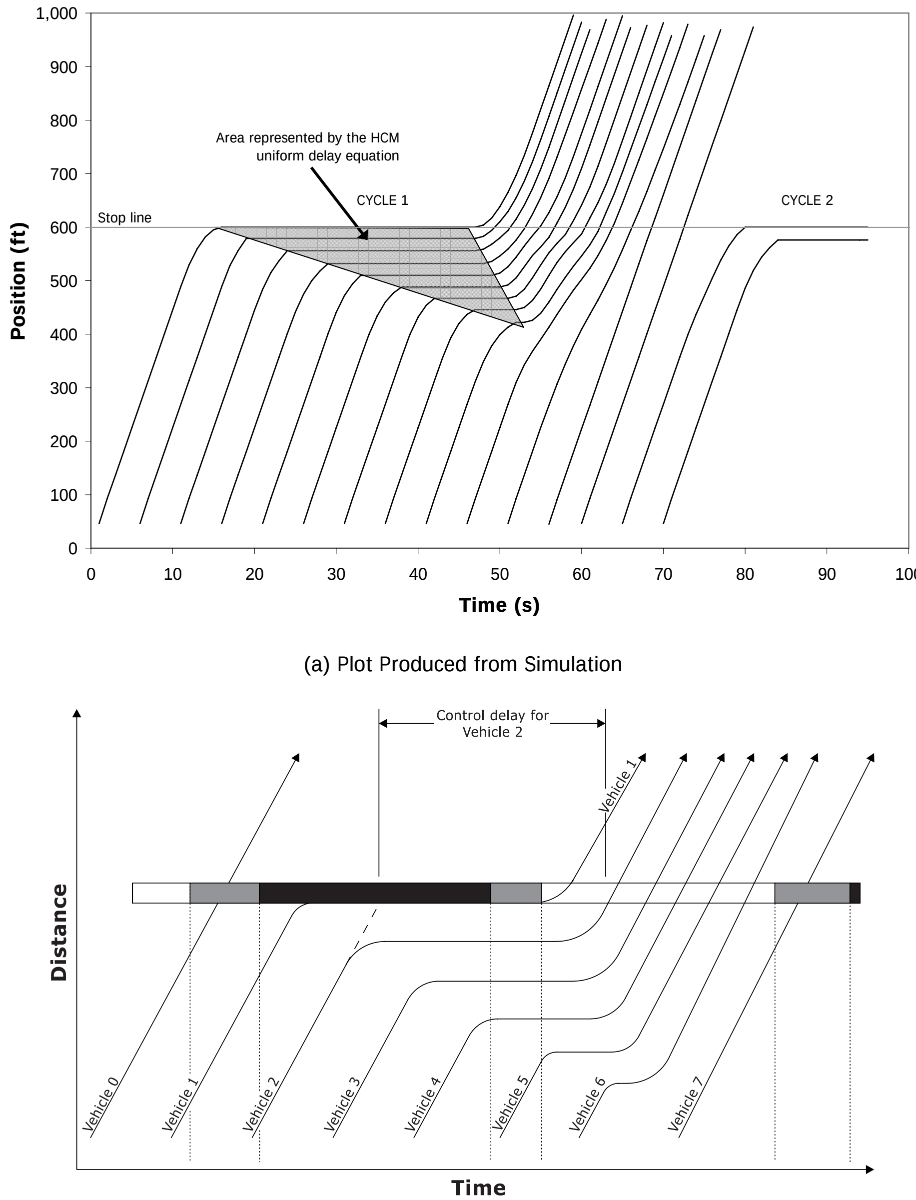
Exhibit 24-3
Trajectory Plots for Uniform Arrivals and Departures

LIVE GRAPH
Click here to view

Note the similarity between the trajectories obtained from the file (above) and those that were developed manually in Chapter 31 (below) to illustrate the basic principles of signalized intersection operation.



(a) Plot Produced from Simulation

(b) Plot Produced by Hand

Exhibit 24-4 shows a sample trajectory plot for the same operation depicted in Exhibit 24-3. As expected, the individual trajectories follow the same pattern as the uniform case, except that some spacings and speeds are not as consistent. The trajectory lines do not cross each other in this example because the example uses a single-lane approach and overtaking is not possible.

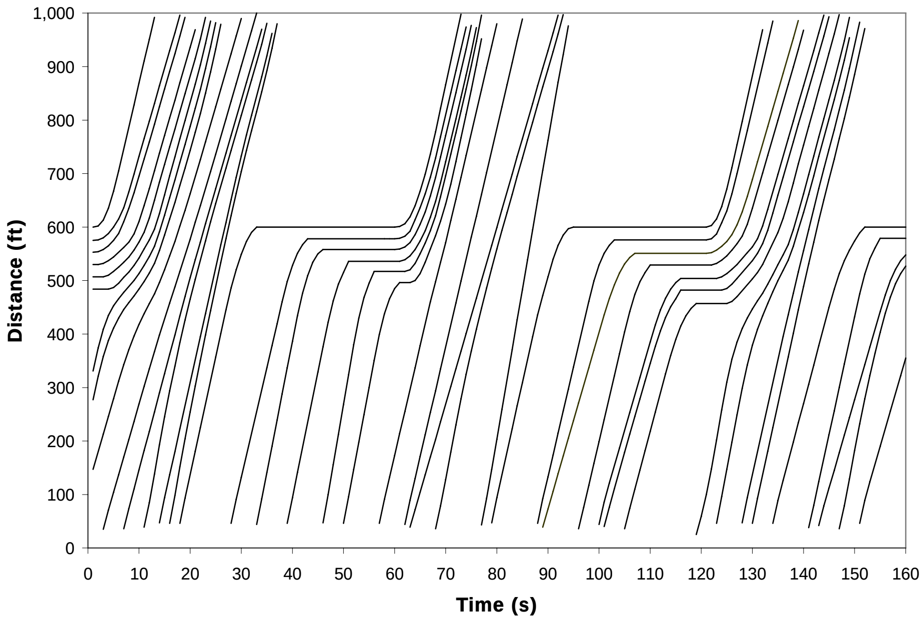


Exhibit 24-4
Introducing Randomness into the Simulation

LIVE GRAPH
Click here to view

Vehicle Trajectories for Oversaturated Operation

Up to this point, the examples have involved volume/capacity (*v/c*) ratios less than 1.0, in which all vehicles arriving on a given cycle were able to clear on the same cycle. Saturation levels close to and above 1.0 present a different picture. Three cases are presented here:

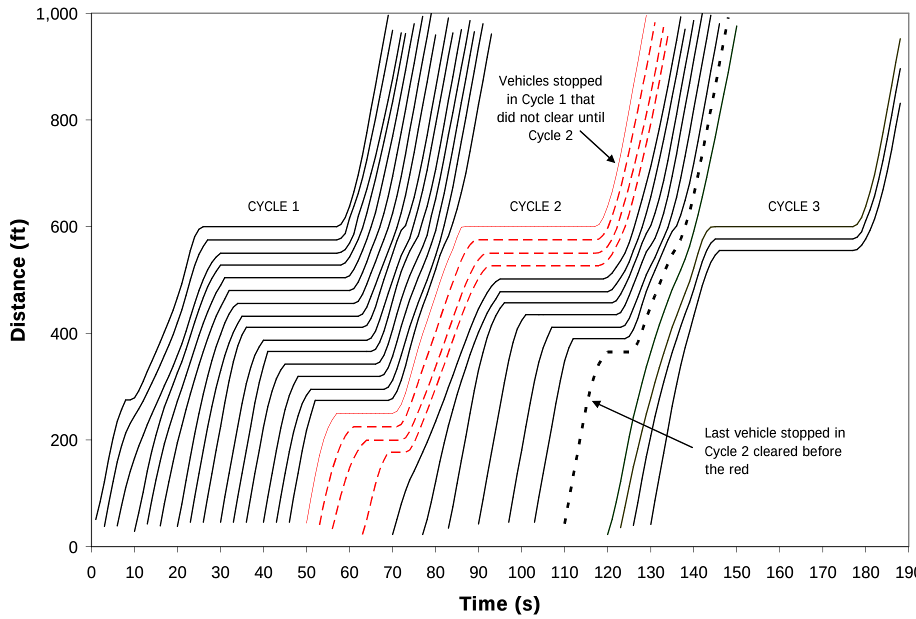
1. *Cycle failure,* occurring when saturation approaches 1.0 and residual queues build on one cycle but are resolved on the next cycle;
2. *Oversaturated operation,* a situation in which the link has a demand volume exceeding the link's capacity and queues extend throughout the approach link; and
3. *Undersaturated operation,* in which queues extend to an upstream link for a part of a cycle because of closely spaced intersections.

Cycle Failure

A cycle failure example is presented in Exhibit 24-5. This trajectory plot shows a situation in which some vehicles arriving in Cycle 1 were unable to clear until Cycle 2. This condition is identified from the trajectory plot for four stopped vehicles (i.e., horizontal trajectory lines) that were forced to stop again before reaching the stop line. These vehicles became the first four vehicles in the queue for Cycle 2. Fortunately, the arrivals during Cycle 2 were few enough that all stopped vehicles were able to clear the intersection before the beginning of the red phase. A closer inspection of Exhibit 24-5 shows that one more vehicle, which was not stopped, was also able to clear.

Exhibit 24-5
Cycle Failure Example

LIVE GRAPH
Click here to view



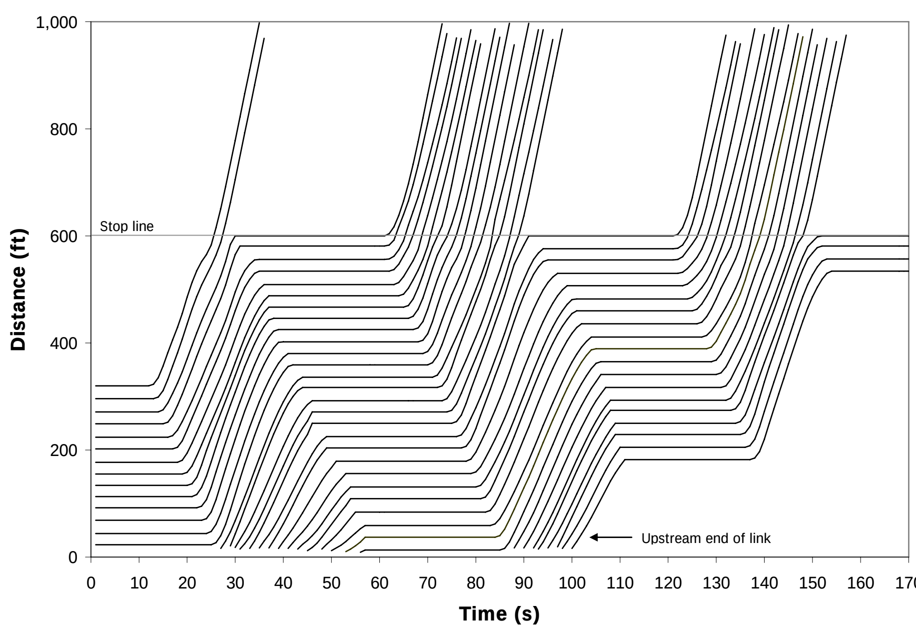
Severely Oversaturated Operation

Oversaturated operation was produced by increasing the demand volume to the point where it exceeded the capacity of the approach. The increased demand produced a queue that extended the length of the link. Inspection of the animated graphics showed that the queue did, in fact, back up beyond the link entry point.

The vehicle trajectory plot for this operation is presented in Exhibit 24-6. The move-up process is represented in the trajectories. Vehicles entering the link require up to three cycles to clear the intersection. The implications on control delay computations when the queue occupies a substantial proportion of the link are discussed in Chapter 7.

Exhibit 24-6
Oversaturated Signal Approach

LIVE GRAPH
Click here to view



A larger question is what to do with the vehicles denied entry during the analysis period. The answer is that, as indicated in Chapter 18, Signalized Intersections, the analysis period must be long enough to include a period of uncongested operation at each end. The delay to vehicles denied entry to this link will be accounted for in upstream links during the period. The upstream links must include a holding area outside the system. Some tools include the delay to vehicles denied entry and some do not. If a tool is used that does not include denied-entry delay, then fictitious links must be built into the network structure for that purpose.

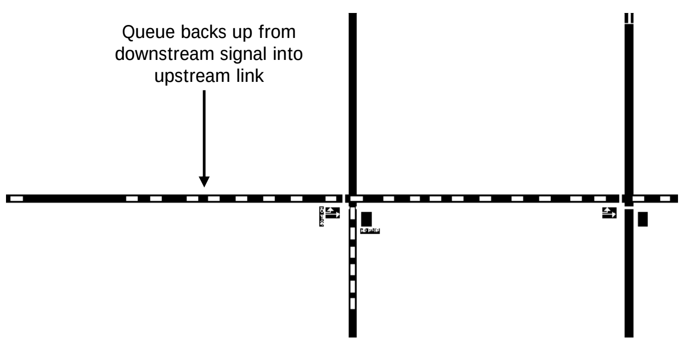
Queue Backup from a Downstream Signal

Even when an approach is not fully saturated, queues might back up from a downstream signal for a portion of the cycle. It happens when intersections are closely spaced. An example of queue backup within a cycle is shown in Exhibit 24-7.

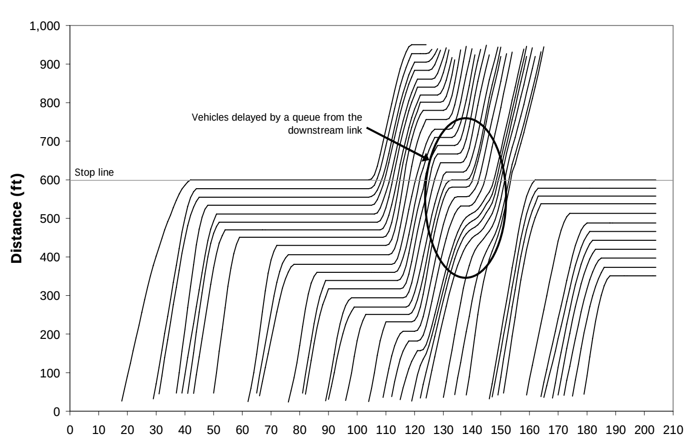
The two-intersection configuration for this example is shown in Exhibit 24-7(a). The graphics screen capture shows that vehicles that would normally pass through the upstream link are prevented from doing so by queues that extend beyond the end of the downstream link for a portion of the cycle. The question is how to treat the resulting delay.

Exhibit 24-7
Queue Backup from a Downstream Signal

LIVE GRAPH
Click here to view



(a) Simulation Graphics Representation



(b) Vehicle Trajectory Representation

By the definitions given to this point, the delay in the upstream link would be assigned to the upstream link, even though the signal on the downstream link was the primary cause. The important thing is not to overlook any delay and to assign all delay somewhere and in a consistent manner. With simulation modeling, the only practical place to assign delay consistently is the link on which the delay occurred. Subtle complexities make it impractical to do otherwise. For example, the root cause of a specific backup might not be the immediate downstream link. In fact, the backup might be secondary to a problem at some distant location in the network at some other point in time.

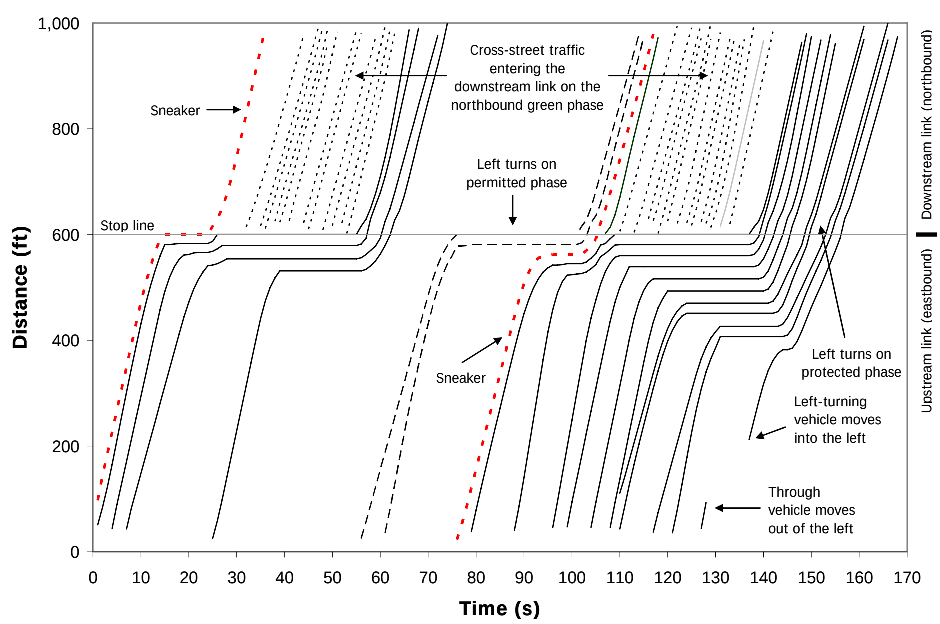
More Complex Signal Phasing

Up to this point only simple signal phasing has been considered. Many applications involve simulating more complex phasing on urban streets. As an example of a more complex situation, a left turn moving on both a protected and a permitted phase is examined.

Exhibit 24-8 shows the trajectory plot for an eastbound left-turn movement from an exclusive lane, controlled by a signal with both protected and permitted phases. In this case, the upstream link is the eastbound approach to the intersection and the downstream link is the northbound approach to the next intersection. Because the distance on a trajectory plot is one dimensional, the distance scale is linear, even though the actual route takes a right-angle bend.

Exhibit 24-8
Trajectory Plot for More Complex Signal Phasing

LIVE GRAPH
Click here to view



Even with an undersaturated operation, this trajectory plot is substantially more involved than the previous ones. Several phenomena are identified in the figure, including the following:

1. Cross-street traffic entering the downstream link on the northbound phase: These vehicles do not appear on the upstream link because they are on a different link. They enter the downstream link at the stop line on the red phase for the left-turn movement of interest.

2. Left turns on the protected phase, shown as solid lines on the trajectory plot: The protected left-turn phase takes place immediately after the red phase. The left-turning vehicles begin to cross the stop line at that point.
3. Left turns on the permitted phase, shown as broken lines on the trajectory plot: The permitted left-turn phase takes place immediately after the protected phase. There is a gap in the trajectory plot because the left-turning vehicles must wait for oncoming traffic to clear.
4. Left-turn “sneakers”: It is not possible to identify a sneaker explicitly on the trajectory plot; however, the last left turn to clear the intersection on the permitted phase is probably a sneaker if it enters at the end of the permitted phase.
5. Left-turn vehicles that enter the link in the through lane and change into the left lane somewhere along the link: These vehicles are identified by trajectories that begin in the middle of the link.
6. Through vehicles that enter the link in the left-turn lane and change into the through lane somewhere along the link: These vehicles are identified by trajectories that end abruptly in the middle of the link.

The trajectory plot shown for this example is more complex than the previous plots; however, performance can be analyzed in the same way.

Freeway Examples

Freeway trajectories follow the same definitions as surface street trajectories, but the queuing patterns differ because they are created by car-following phenomena and not by traffic signals. The performance measures of interest also differ. There is no notion of control delay on freeways because there is no control. The level of service on uninterrupted-flow facilities is based on traffic density expressed in units of vehicles per mile per lane. In some cases, such as merging segments, the density in specific lanes is of interest.

Two cases are examined. The first case deals with a weaving segment, and the second case deals with merging at an entrance ramp.

Weaving Segment Example

Simulation Network Structure

The problem description, link–node structure, and animated graphics view for the weaving segment example are shown in Exhibit 24-9. It is the same scenario used in Example Problem 1 in Chapter 12, Freeway Weaving Segments. There are two lanes on the freeway and on each ramp. The two ramp lanes are connected by full auxiliary lanes.

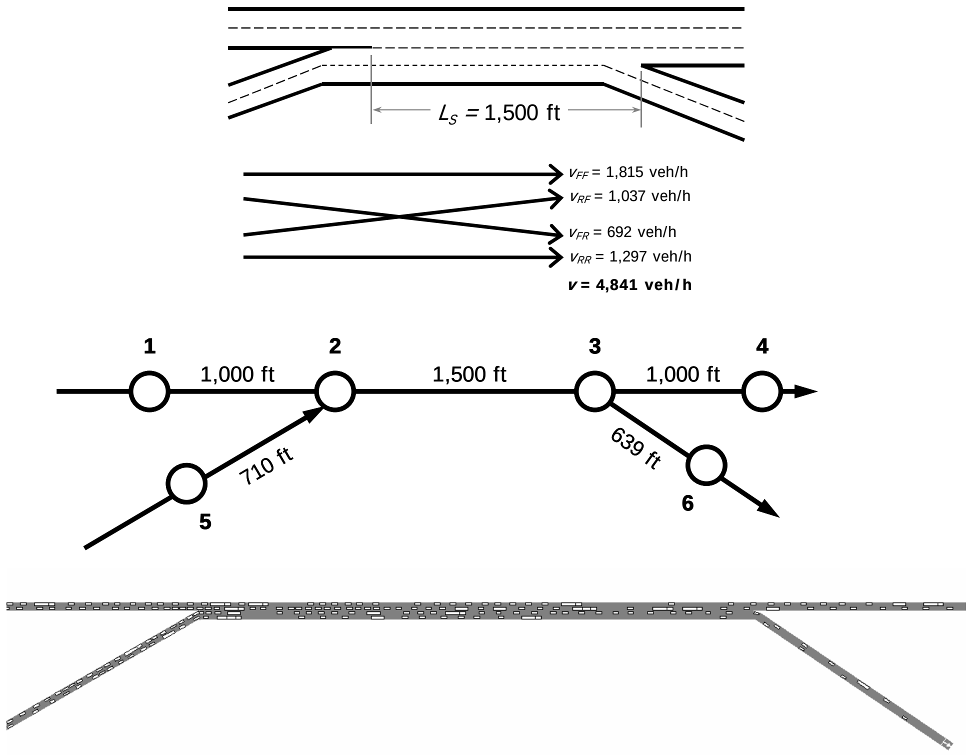


Exhibit 24-9
Weaving Segment Description and Animated Graphics View

Note: L_S = length of segment, V_{FF} = vehicles entering from freeway and leaving to freeway, V_{RF} = vehicles entering from ramp and leaving to freeway, V_{FR} = vehicles entering from freeway and leaving to ramp, V_{RR} = vehicles entering from ramp and leaving to ramp, veh/h = vehicles per hour.

Vehicle Trajectories for the Freeway Lanes

The vertical (i.e., distance) axis of the trajectory plot provides a linear one-dimensional representation of a series of connected links. The links can follow any pattern as long as some of the vehicles leaving one link flow into the next link. The VTAPE utility accommodates a maximum of eight connected links. When multiple links are connected to a node (as is usually the case), then different combinations of links may be used to construct a multilink trajectory analysis. The route configuration must be designed with the end product in mind. Sometimes it is necessary to examine multiple routes to obtain a complete picture of the operation.

There are two entry links and two exit links to the weaving segment, giving four possible routes for analysis. Two routes are examined in this example. The first route, which is represented in Exhibit 24-10, shows the traffic entering the weaving segment from the freeway and leaving to the freeway (V_{FF} in Exhibit 24-9), represented by Links 1–2–3–4.

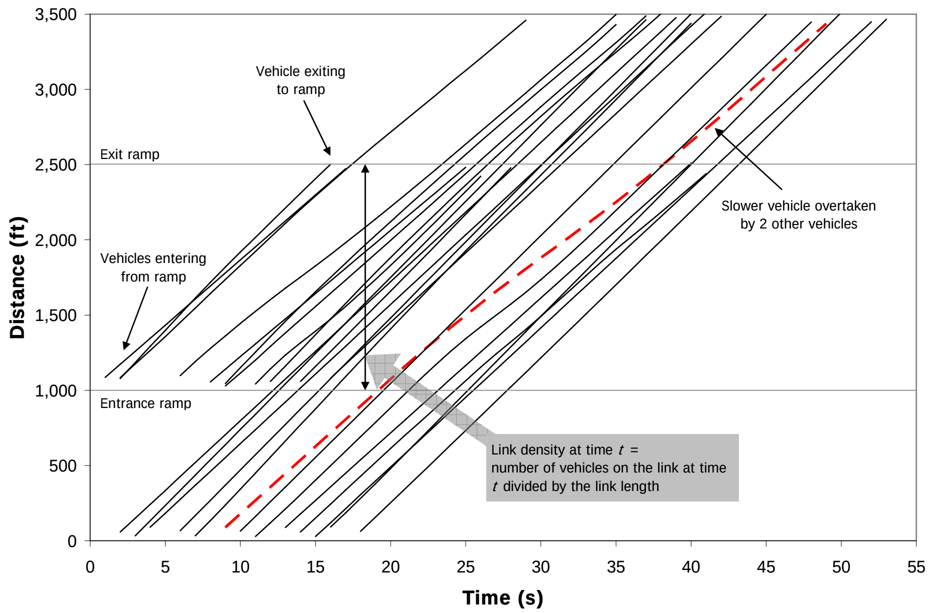


Exhibit 24-10
Trajectory Plot for Freeway Links

LIVE GRAPH
Click here to view

This plot is a multilane plot so, unlike previous plots, some of the trajectory lines might cross each other because of different speeds in different lanes. One such instance is highlighted in Exhibit 24-10. This figure also shows vehicles that enter and leave the weaving segment on the ramps. Because the ramps are not part of the selected route, the ramp vehicles appear on the trajectory plot only on the link that represents the weaving segment. Examples of ramp vehicles are identified in the figure.

The definition of link density (vehicles per mile) is also indicated in Exhibit 24-10. Density as a function of time *t* is expressed in vehicles per mile and is determined by counting the number of vehicles within the link and dividing by the link length in miles. Average lane density (vehicles per mile per lane) on the link may then be determined by dividing the link density by the number of lanes. To obtain individual lane densities, it is necessary to perform the trajectory analysis on each lane. The analysis must also be performed on a per lane basis to examine individual vehicle headways.

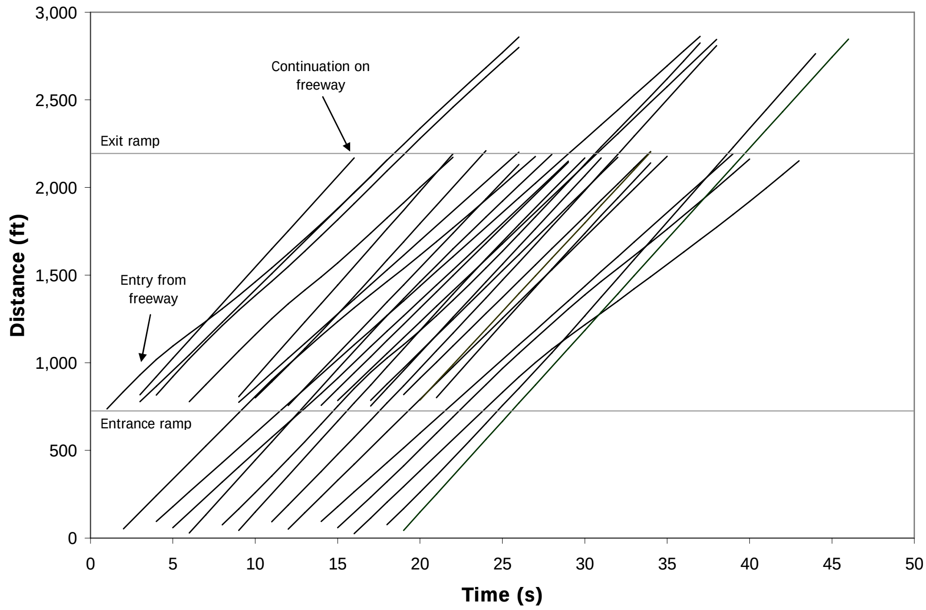
Vehicle Trajectories for the Entrance and Exit Ramps

By specifying the links on the route as 5–2–3–6 instead of 1–2–3–4, the trajectories for vehicles entering and leaving the weaving segment on the ramps (V_{RR} in Exhibit 24-9) can be examined. This trajectory plot is shown in Exhibit 24-11. This figure is very similar to Exhibit 24-10, except that the vehicles that do not appear outside the weaving segment are those on the freeway links instead of the ramp links.

It is also possible to construct two other routes, one for vehicles entering from the freeway and leaving to the exit ramp, V_{FR}, as 1–2–3–6, and for those entering from the ramp and leaving to the freeway, V_{RF}, as 5–2–3–4. These plots are not included here.

Exhibit 24-11
Trajectory Plot for Entrance and Exit Ramp Links

LIVE GRAPH
Click here to view



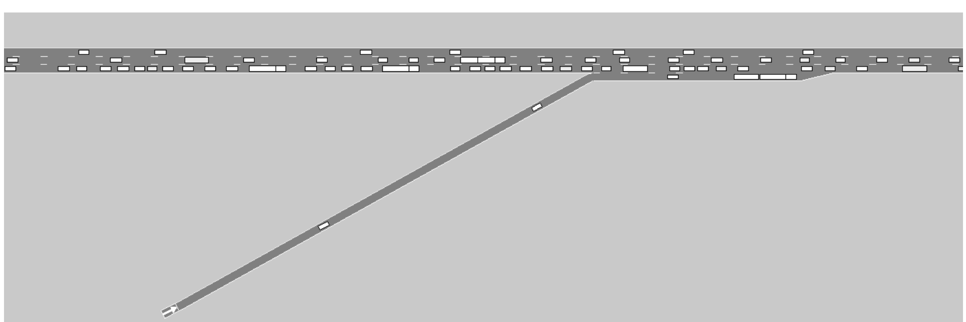
Entrance Ramp Merging Example

Merging segments provide another good example of vehicle trajectory analysis on a freeway. The merging vehicles affect the freeway operation differently in each lane, so it is necessary to examine each lane independently.

Simulation Network Structure

The same node structure used in the weaving segment example is used here. The lane configuration has been changed to be more representative of a merge operation. Three lanes have been assigned to the freeway and one lane to the entrance ramp. The demand volumes have been specified to provide a near-saturated operation to observe the effects of merging under these conditions. A graphic view of the operation is presented in Exhibit 24-12.

Exhibit 24-12
Entrance Ramp Merging Segment Graphics View

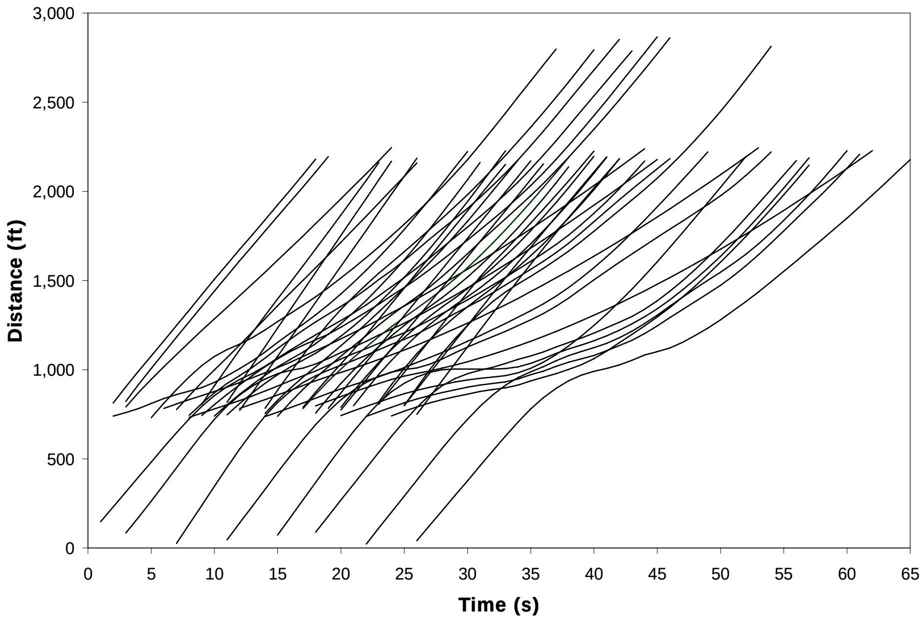


Trajectory Plots for All Lanes

Exhibit 24-13 shows the trajectory plot for all freeway lanes combined. The operation is clearly heterogeneous, with a mixture of fast and slow speeds. Many trajectory lines cross each other, and not much can be done in the way of analysis with the data from this scenario.

Exhibit 24-13
Trajectory Plot for All Freeway Lanes in the Merge Area

LIVE GRAPH
Click here to view



Trajectory Plots for Individual Lanes

It is clearly necessary to examine each lane individually. Exhibit 24-14, Exhibit 24-15, and Exhibit 24-16 show the trajectory plots for Lanes 1, 2, and 3, respectively. Because these plots represent individual lanes, the trajectory lines do not cross each other. The effect of the merging operation is observable (and predictable) in these three figures.

Exhibit 24-14
Trajectory Plot for Freeway Lane 1 (Rightmost) in the Merge Area

LIVE GRAPH
Click here to view

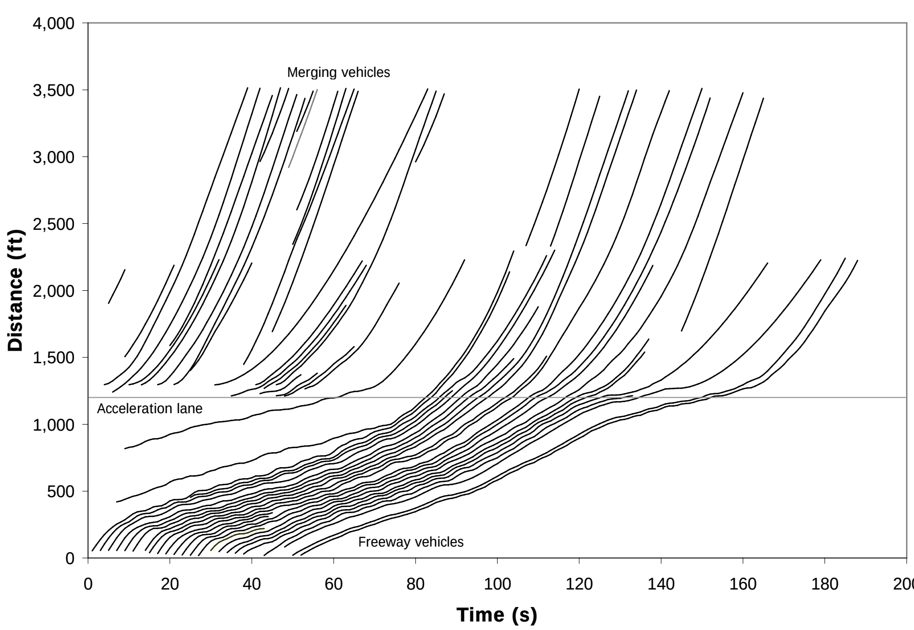


Exhibit 24-15
Trajectory Plot for Freeway Lane 2 (Center) in the Merge Area

LIVE GRAPH
Click here to view

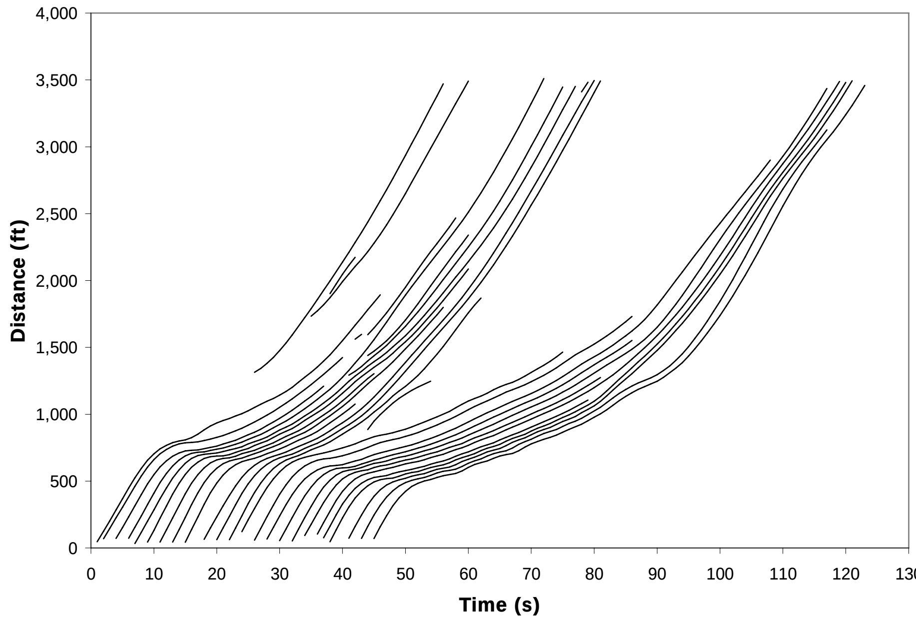
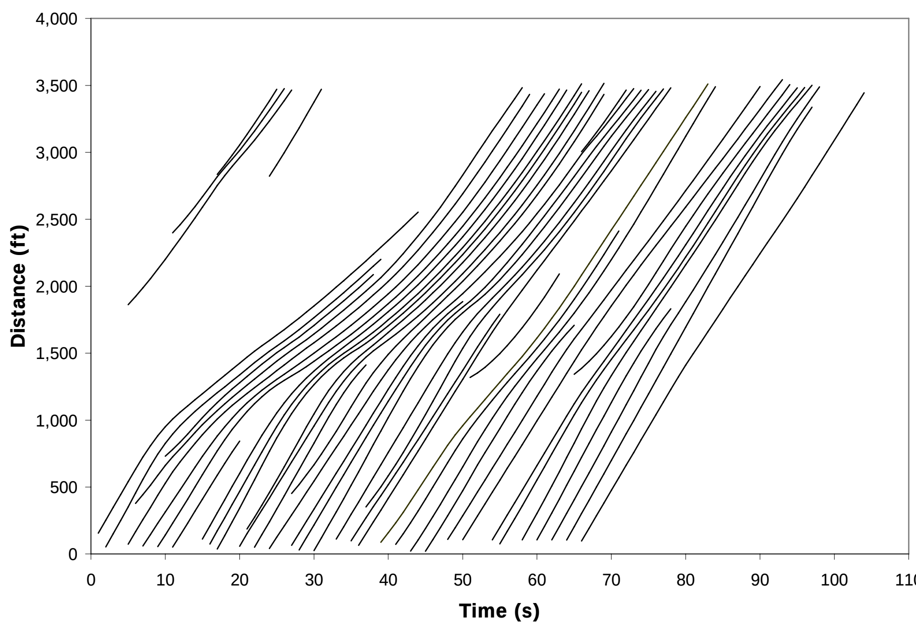


Exhibit 24-16
Trajectory Plot for Freeway Lane 3 (Leftmost) in the Merge Area

LIVE GRAPH
Click here to view



In Lane 1, freeway speeds are low upstream of the merge point. Merging vehicles enter the freeway slowly but pick up speed rapidly downstream of the merge point bottleneck. The merging vehicles enter the freeway from the acceleration lane, which begins at 1,000 ft on the distance scale. The merging vehicle trajectories before entering the freeway are not shown in Exhibit 24-14 because those vehicles are either on a different link or in a different lane.

In Lane 2, the freeway speeds are higher but still well below the free-flow speed, indicating that the merge operation affects the second lane as well. Some vehicles enter Lane 2 in the vicinity of the acceleration lane, but they are generally vehicles that have left Lane 1 to avoid the friction. Both Lane 1 and Lane 2 show several discontinuous trajectories that indicate lane changes. The

Lane 3 operation is much more homogeneous and speeds are higher, indicating a much smaller effect of the merging operation.

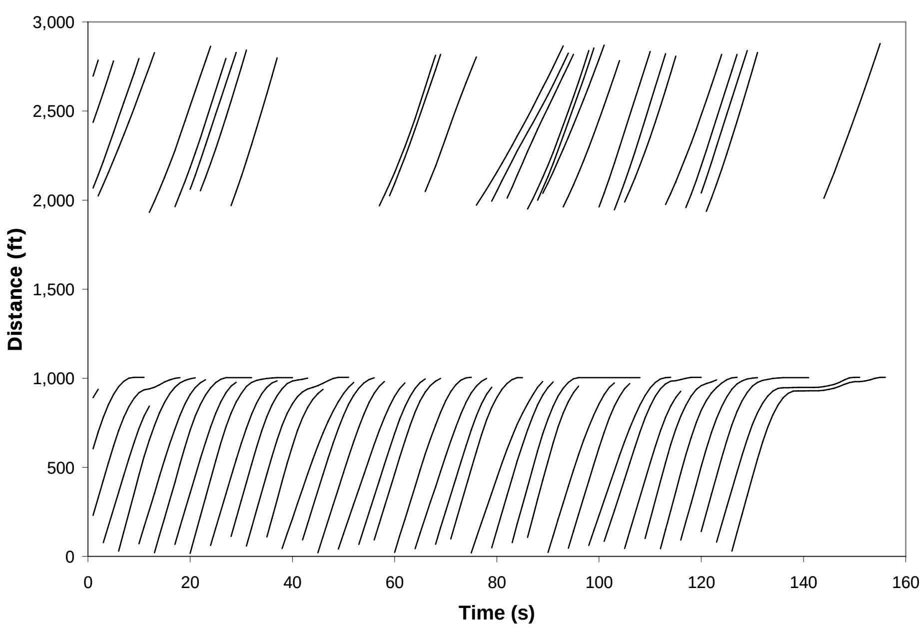
Trajectory Plots for Ramp Vehicles

To configure a trajectory route covering the entrance ramp vehicles, the ramp and acceleration lane, which were not represented in Exhibit 24-14 through Exhibit 24-16, must be selected in place of the upstream freeway link. The acceleration lane number must first be identified from the simulation tool's output. Because of the selected tool's unique and somewhat creative lane numbering scheme, the acceleration lane will be Lane 9. To cover both the ramp and the acceleration lane, Lane 9 must be selected on the freeway link (2–3).

The trajectory plot for this route is shown in Exhibit 24-17. The results are not quite what one might anticipate. Vehicles are observed on the ramp and in the acceleration lane, but they disappear as soon as they enter the freeway. More vehicles eventually appear toward the end of the freeway link. The vehicles disappear because Lane 9 was selected for the freeway link, so vehicles in Lane 1 do not show up on the plot. The vehicles that reappear at the end of the link are those that are leaving the freeway at the downstream exit. They reappear at that point because the deceleration lane at the end of the link is also assigned as Lane 9. This plot is not particularly useful, except that it illustrates the complexities of trajectory analysis.

Exhibit 24-17
Trajectory Plot for Acceleration and Deceleration Lanes

LIVE GRAPH
Click here to view



To obtain a continuous plot of ramp vehicles, nodes must be added to the network at the points where the acceleration and deceleration lanes join the freeway. These nodes are shown as Nodes 7 and 8 in Exhibit 24-18. A continuous route may then be configured as 5–2–7–8–3–4. The trajectory plot for this route is shown in Exhibit 24-19. This plot shows the entering vehicles on the ramp as they pass through the acceleration lane onto the freeway. There are some

discontinuities in the trajectories because of the different point at which vehicles leave the acceleration lane.

Exhibit 24-18
Addition of Intermediate Nodes for Continuous Trajectory Plots

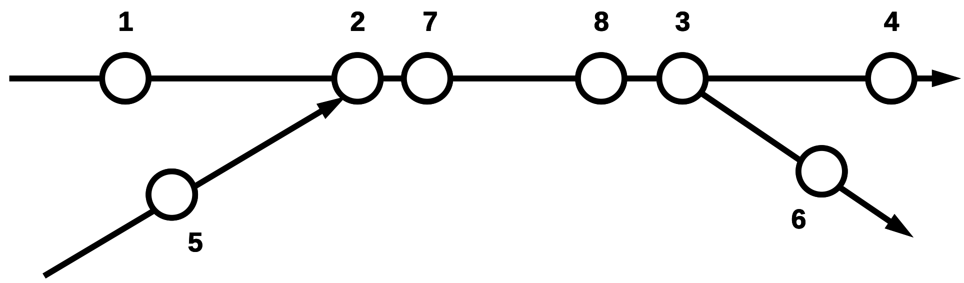


Exhibit 24-19
Trajectory Plot for Acceleration Lane and Freeway Lane 1

LIVE GRAPH
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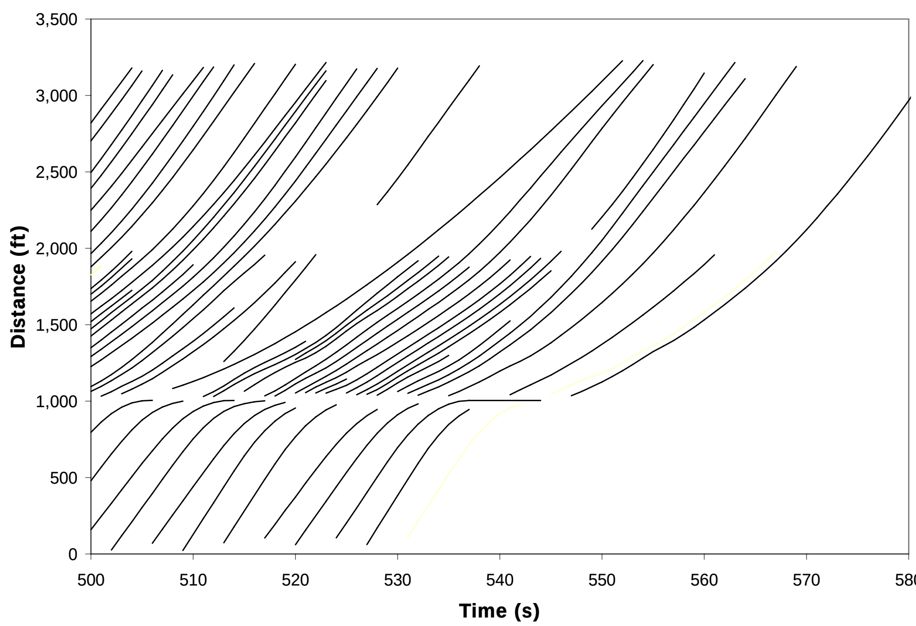

ESTIMATING PERFORMANCE MEASURES FROM VEHICLE TRAJECTORY DATA

The preceding subsections demonstrated that it is possible to produce vehicle trajectory plots that can be interpreted and analyzed. This subsection focuses more on computation of the performance measures from a mathematical analysis of the data represented in these plots.

Trajectory Analysis Procedures Overview

One development goal for this edition of the HCM was the creation of a set of computational procedures by which developers of simulation tools could produce performance measures that are consistent among different tools and, to the extent possible, compatible with the HCM's deterministic procedures. The procedures presented here were designed to be implemented easily by using the common trajectory properties described previously and illustrated by examples. Developers of simulation tools are encouraged to implement these procedures, and users of simulation tools are encouraged to consider the extent to which the procedures have been implemented in the traffic analysis tool selection process described in Chapter 6, HCM and Alternative Analysis Tools.

Requirements for Trajectory Analysis Algorithm Development

A basic set of guidelines for computing uniform performance measures from vehicle trajectory analysis was introduced in Chapter 7, Interpreting HCM and Alternative Tool Results. Since these requirements are also incorporated into the specific computational procedures proposed in this chapter, they are repeated here to promote a better understanding of the procedures. The general guidelines proposed in Chapter 7 include the following:

1. The trajectory analysis procedures are limited to analysis of trajectories produced by the traffic flow model of each simulation tool. The nature of the procedures must not suggest the need for developers to change their driver behavior or traffic flow modeling logic.
2. If the procedures for estimating a particular measure cannot be satisfactorily defined to permit a valid comparison between the HCM and other modeling approaches, then such comparisons should not be made.
3. All performance measures that accrue over time and space shall be assigned to the link and time interval in which they occur. Subtle complexities make it impractical to do otherwise. For example, the root cause of a specific delay might not be within the link or the immediate downstream link. In fact, the delay might be secondary to a problem at some distant location in the network and in a different time interval.
4. The analyst must understand that the spatial and temporal boundaries of the analysis domain must include a period that is free of congestion on all sides. This principle is also stated in Chapter 10, Freeway Facilities, and in Chapter 18, Signalized Intersections, for multiperiod signalized intersection analysis. To ensure that delays to vehicles denied entry to the system during a given period are properly recognized, it might be necessary to create fictitious links outside the physical network to hold such vehicles. A more detailed discussion of spatial and temporal boundaries is provided in Chapter 7.
5. It is important to ensure that the network has been properly initialized or "seeded" before trajectory analysis is performed. When the warm-up periods are set and applied, simulation tools typically start with an empty network and introduce vehicles until the vehicular content of the network stabilizes. Trajectory analysis should not begin until stability has been achieved. If the simulation period begins with oversaturated conditions, stability may never be achieved. See the discussion in Chapter 7 on temporal and spatial boundaries.

In addition to the general guidelines, some requirements must be addressed here to promote the development of trajectory analysis procedures that can be applied in a practical manner by the developers of simulation tools. The following requirements are suggested:

1. The algorithms must be suitable for computation "on the fly." They must not require information from a future time step that would complicate the data handling within the simulation process.

2. Arbitrary thresholds should be kept to a minimum because of the difficulty of obtaining acceptance throughout the user community for specific thresholds. When arbitrary thresholds cannot be avoided, they should be justified to the extent possible by definitions in the literature and, above all, they should be applied consistently for different types of analysis.
3. Computationally complex and time-consuming methods should be avoided to minimize the additional load on the model. Methods should be developed to simplify situations with many special cases because of the difficulty of enumerating all special cases.
4. The same definitions, thresholds, and logic should be used for determination of similar parameters in different computational algorithms for longitudinal and spatial analysis.

Summary of Computational Procedures

Several performance measures were examined in Chapter 7, and general guidelines for comparing measures produced by different tools were presented. Previous material in this section has demonstrated the potential for development of uniform measures by individual vehicle trajectory analysis and has proposed some requirements for development of the analysis procedures. Specific procedures for analyzing vehicle trajectories are now presented and demonstrated with additional examples.

Thresholds for Computation of Performance Measures

Elimination of arbitrary and user-specified values is an important element of standardization. Avoidance of arbitrary thresholds was identified earlier as a requirement for the development of trajectory analysis procedures. It is desirable to avoid all arbitrary thresholds. If thresholds cannot be avoided, they should be justified in terms of the literature. When no such justification exists, they should at least be established on the basis of consensus and applied consistently. The following thresholds cannot be avoided in vehicle trajectory analysis.

Car Length

The following is stated in Chapter 31, Signalized Intersections: Supplemental:

> *A vehicle is considered as having joined the queue when it approaches within one car length of a stopped vehicle and is itself about to stop. This definition is used because of the difficulty of keeping precise track of the moment when a vehicle comes to a stop.*

So, for estimation of queue-related measures, it is necessary to choose a value that represents one car length. For the purposes of this section, a value of 20 ft is used.

Stopped-Vehicle State

One example of an arbitrary threshold is the speed at which a vehicle is considered to have come to a stop. Several arbitrary thresholds have been

applied for this purpose. To maintain consistency with the definition of the stopped state applied in other chapters of the HCM, a speed less than 5 mi/h is used here for determining when a vehicle has stopped.

Moving-Vehicle States

Other states in addition to the stopped state that must be defined consistently for vehicle trajectory analysis include the following:

- The uncongested state, in which a vehicle is moving in a traffic stream that is operating below its capacity;
- The congested state, in which the traffic stream has reached a point that is at or slightly above its capacity, but no queuing from downstream bottlenecks is present; and
- The severely constrained state, in which downstream bottlenecks have affected the operation.

These states apply primarily to uninterrupted flow. A precise definition would require very complex modeling algorithms involving capacity computations or "look ahead" features, both of which would create a computational burden. Therefore, an easily applied approximation must be sought. Threshold speeds are a good candidate for such an approximation.

It is convenient to think of these states in terms of speed ranges. To avoid specifying arbitrary speeds as absolute values, it is preferable to use the target speed of each vehicle as a reference. The target speed is the speed at which the driver prefers to travel. It differs from the free-flow speed in the sense that most simulation tools apply a "driver aggressiveness" factor to the free-flow speed to determine the target speed. In the absence of accepted criteria, three equal speed ranges are applied for the purposes of this section. Thus, the operation is defined as uncongested if the speed is above two-thirds of the target speed. It is defined as severely constrained when the speed is below one-third of the target speed, and it is considered congested in the middle speed range. This stratification is used to produce performance measures directly (e.g., percent of time severely constrained). It is also used in computing other performance measures (e.g., release from a queue).

Computational Procedures for Stop-Related Measures

The two main stop-related measures are number of stops and stopped delay. The beginning of a stop is defined in the same way for both measures. The end of a stop is treated differently for stopped delay and number of stops. For stopped delay, the end of a stop is established as soon as the vehicle starts to move (i.e., its speed reaches 5 mi/h or greater). For determining the number of stops, some hysteresis is required. For purposes of this section, after a vehicle is stopped a subsequent stop is not recognized until it leaves the severely constrained state (i.e., its speed reaches one-third of the target speed).

Because subsequent stops are generally made from a lower speed, they can be expected to have a smaller impact on driver perception, operating costs, and safety. Recognizing this fact, the NCHRP 3-85 project proposed a "proportional stop" concept (*1*), in which the proportion of a subsequent stop is based on the

relative kinetic energy loss and is therefore proportional to the square of the speed from which the stop was made. Thus, each time a vehicle speed drops below 5 mi/h, the number of stops is incremented by $(S_{max}/S_{target})^2$, where S_{max} is the maximum speed attained since the last stop and S_{target} is the target speed.

While this procedure has not been applied in practice, it is mentioned here because it offers an interesting possibility for the use of simulation to produce measures that could be obtained in the field but could not be estimated by the macroscopic deterministic models described in the HCM. The procedure is illustrated by an example later in this section.

Computational Procedures for Delay-Related Measures

The procedures for computing delay from vehicle trajectories involve aggregating all delay measures over each time step. Therefore, the results take the form of aggregated delay and not unit delay, as defined in Chapter 7. To determine unit delays, the aggregated delays must be divided by the number of vehicles involved in the aggregation. Partial trips made over a segment during the time period add some complexity to the unit delay computations.

The following procedures should be used to compute the various delay-related measures from vehicle trajectories:

- *Time step delay*: The delay on any time step is, by definition, the length of the time step minus the time it would have taken the vehicle to cover the distance traveled in the step at the target speed. This value is easily determined and is the basis for the remainder of the delay computations.
- *Segment delay*: Segment delay is represented by the time actually taken to traverse a segment minus the time it would have taken to traverse the segment at the target speed. The segment delay on any step is equal to the time step delay. Segment delays accumulated over all time steps in which a vehicle is present on the segment represent the segment delay for that vehicle.
- *Queue delay*: Queue delay is equal to the time step delay on any step in which the vehicle is in a queued state; otherwise, it is zero. Queue delays are accumulated over all time steps while the vehicle is in a queue.
- *Stopped delay:* Stopped delay is equal to the time step delay on any step in which the vehicle is in a stopped state; otherwise, it is zero. Because a vehicle is considered to be "stopped" if it is traveling at less than a threshold speed, a consistent definition of stopped delay requires that the travel time at the target speed be subtracted. Time step delays accumulated over all time steps in which the vehicle was in the stopped state represent the stopped delay.

Queue delay computed from trajectory analysis provides the most appropriate representation of control delay.

- *Control delay*: Control delay is the additional travel time caused by operation of a traffic control device. It cannot be computed directly from the vehicle trajectories in a manner consistent with the procedures given in Chapters 18 and 31 for signalized intersection analysis. It is, however, an important measure because it is the basis for determining the level of service on a signalized approach.

The queue delay computed from vehicle trajectories provides a reasonable approximation of control delay when the following conditions are met:

1. The queue delay is caused by a traffic control device, and
2. The identification of the queued state is consistent with the definitions provided in this section.

Computational Procedures for Queue-Related Measures

Procedures for computing queue-related measures begin with determining whether each vehicle in a segment is in a queued state. A vehicle is in a queued state if it has entered a queue and has not yet left it. The beginning of a queued state occurs when

- The gap between a vehicle and its leader is less than or equal to 20 ft,
- The vehicle speed is greater than or equal to the leader speed, and
- The vehicle speed is less than or equal to one-third of the target speed (i.e., the speed is severely constrained).

A separate case must be created to accommodate the first vehicle to arrive at the stop line. If the link is controlled (interrupted-flow case), the beginning of the queued state also occurs when

- No leader is present on the link,
- The vehicle is within 50 ft of the stop line, and
- The vehicle is decelerating or has stopped.

These rules have been found to cover all the conditions encountered.

The ending of the queued state also requires some rules. For most purposes, the vehicle should be considered to remain in the queue until it leaves the link. The analysis is done on a link-by-link basis. In the case of queues that extend over multiple links, a vehicle leaving a link immediately enters the queue on the next link. Experience with trajectory analysis has shown that other conditions need to be applied to supplement this rule. Thus, the end of the queued state also occurs when

- The vehicle has reached two-thirds of the target speed (i.e., uncongested operation), and
- The leader speed is greater than or equal to the vehicle speed or the vehicle has no leader in the same link.

The additional conditions cover situations in which, for example, a vehicle escapes a queue by changing lanes into an uncongested lane (e.g., through vehicle caught temporarily in a turn bay overflow).

Chapters 18 and 31 offer the following guidance on estimating queue length:

1. The maximum queue reach (i.e., back of queue, or BOQ) is a more useful measure than the number of vehicles in the queue, because the BOQ causes blockage of lanes. The maximum BOQ is reached when the queue has almost dissipated (i.e., has zero vehicles remaining).

2. A procedure is prescribed to estimate average maximum BOQ on a signalized approach.

The BOQ at any time step will be determined by the position of the last queued vehicle on the link plus the length of that vehicle.

Because of its macroscopic nature, the HCM queue estimation procedure cannot be applied directly to simulation. On the other hand, simulation can produce additional useful measures because of its higher level of detail. The first step in queue length determination has already been dealt with by setting up the rules for determining the conditions that indicate when a vehicle is in a queue. The next step is to determine the position of the last vehicle in the queue.

The BOQ on any step is a relatively simple thing to determine. The trick is to figure out how to accumulate the individual BOQ measures over the entire period. Several measures can be produced.

1. The maximum BOQ at some percentile value—for example, 95%;
2. The maximum BOQ on any cycle at some percentile value—for example, 95%;
3. The historical maximum BOQ (i.e., the longest queue recorded during the period;
4. The probability that a queue will back up beyond a specified point; and
5. The proportion of time that the queue will be backed up beyond a specified point.

Some of these measures are illustrated later in an example.

Computational Procedures for Density-Related Measures

The uninterrupted-flow procedures described in the HCM base their level of service estimates on the density of traffic in terms of passenger cars per mile per lane (pc/mi/ln). In one case (freeway merges and diverges), the density is estimated only for the two lanes adjacent to the ramp.

Density computations do not require a detailed analysis of the trajectory of each vehicle. They are best made by simply counting the number of vehicles in each lane on a given segment, recognizing that the results represent actual vehicles and not passenger cars.

For comparable results, it essential to convert the simulated densities to pc/mi/ln, especially if simulation tools are used to evaluate the level of service on a segment. Because the effect of heavy vehicles on the flow of traffic is treated microscopically, there is no notion of passenger car equivalence in simulation modeling. In addition, traffic flow models may differ among the various simulation tools in their detailed treatment of heavy vehicles. Therefore, it is not possible to prescribe a simple conversion process that will ensure full compatibility with the HCM's level-of-service estimation procedures. One possible method to develop passenger-car-equivalence conversion factors involves multiple simulation runs:

1. Use the known demand flow rates, v, and truck proportions to obtain the resulting segment density in vehicles per mile per lane (veh/mi/ln), d_1.
2. Use the known demand flow rates, v, with passenger cars only to obtain the resulting segment density in veh/mi/ln, d_2.

3. Determine the heavy-vehicle equivalence factor as $f_{HV} = d_2/d_1$.
4. Set the demand flow rates to v/f_{HV} with passenger cars only to obtain the resulting segment density in pc/mi/ln.

This process is more precise because it adheres to the definition of passenger car equivalence. Unfortunately, it is too complicated to be of much practical value. There are, however, two methods that could produce a more practical approximation. Both require determining the heavy-vehicle adjustment factor, f_{HV}, by the method prescribed in Chapter 11 for basic freeway segments. This method is also referenced and used in the procedural chapters covering other types of freeway segments. The simplest approximation may be obtained by running the simulation with known demand flow rates and truck proportions and then dividing the simulated density by f_{HV}. Another approximation involves dividing the demand flow rates by f_{HV} before running the simulation with passenger cars only. The resulting densities are then expressed in pc/mi/ln. The second method conforms better to the procedures prescribed in Chapters 11 to 13, but the first method is probably easier to apply.

Follower density is an emerging density-based measure for two-lane highways (*2*, *3*). It is defined as the number of followers per mile per lane. A vehicle can be classified as following when

- The gap between the rear and the front ends of the leading and following vehicles, respectively, are shorter than or equal to 3 s, and
- The speed of the following vehicle is not more than 12 mi/h lower than that of the preceding vehicle.

The follower density can be derived from point measurements by means of the following formula:

$$\text{Follower Density} = \%\ \text{Followers} \times \text{Flow Rate/Time Mean Speed}$$

Although this performance measure is not computed by the procedures in the HCM, it is worth mentioning because it has attracted significant international interest and is a measure that can easily be computed by vehicle trajectory analysis.

Analysis of a Signalized Approach

The simple approach to a signalized intersection (Exhibit 24-2) is now converted to a two-lane approach with a length of 2,000 ft. A 10-min (600-s) analysis period is used. The cycle length is 60 s, giving 10 cycles for inspection. The analysis period would normally be longer, but 10 min is adequate for demonstration purposes.

Trajectory Plots

The trajectory plot for the first few cycles is shown in Exhibit 24-20. The vehicle track selected for later analysis is also shown in this exhibit.

Exhibit 24-20
Trajectories for Several Cycles on a Signalized Approach

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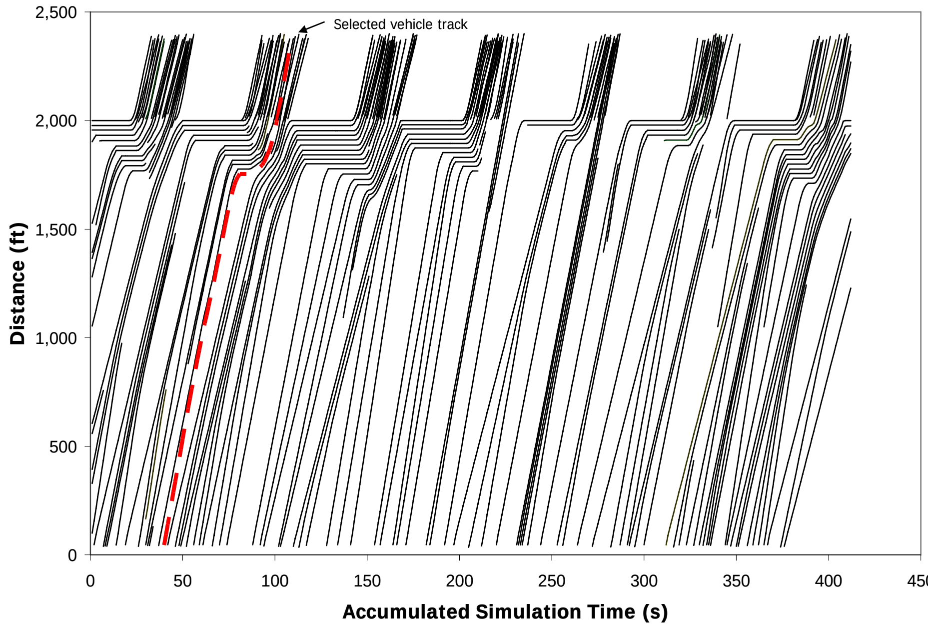

Two individual trajectory analysis plots are shown in Exhibit 24-21. The first plot shows the trajectories of two vehicles where the progress of the subject vehicle is constrained by its leader. The second plot shows the speed and acceleration profiles for the subject vehicle.

Exhibit 24-21
Example Trajectory Analysis Plots

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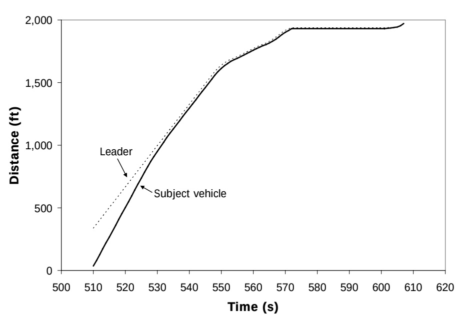
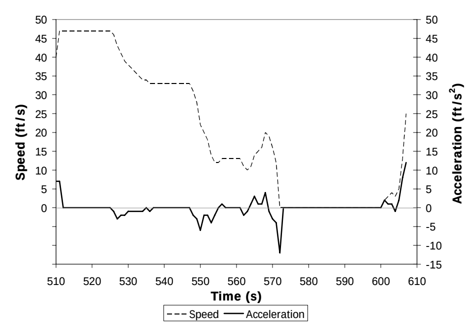


(a) Subject Vehicle and Leader Vehicle Trajectories



(b) Speed and Acceleration Profile of Subject Vehicle

Analysis of Stops

An example of the analysis of a single vehicle selected from the entire trajectory plot is shown in Exhibit 24-22. With the definition of a partial stop based on the NCHRP 3-85 kinetic energy loss concept, the total stop value was 1.81 because the second stop was made from a lower speed.

Exhibit 24-22
Analysis of a Full and Partial Stop

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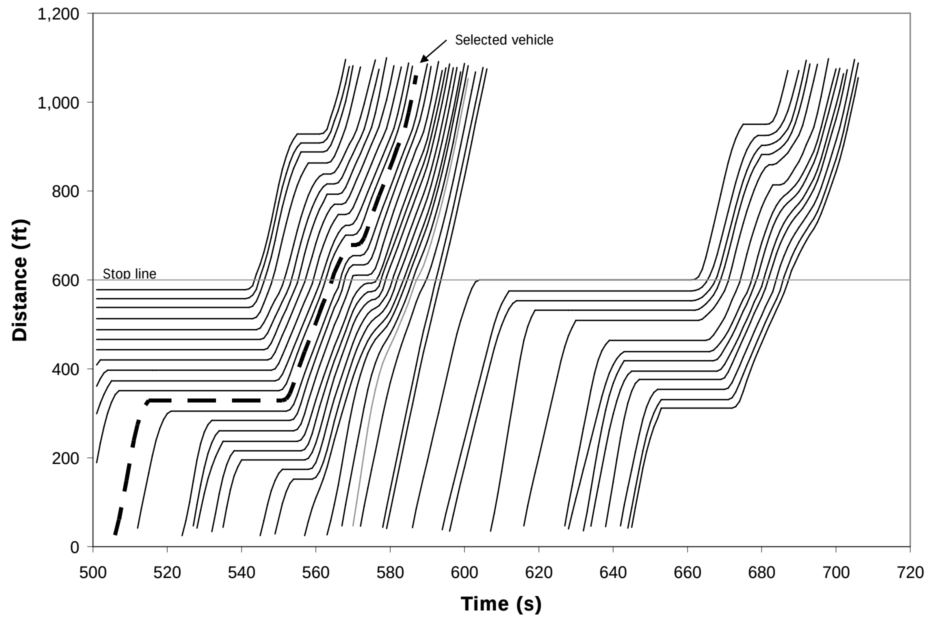
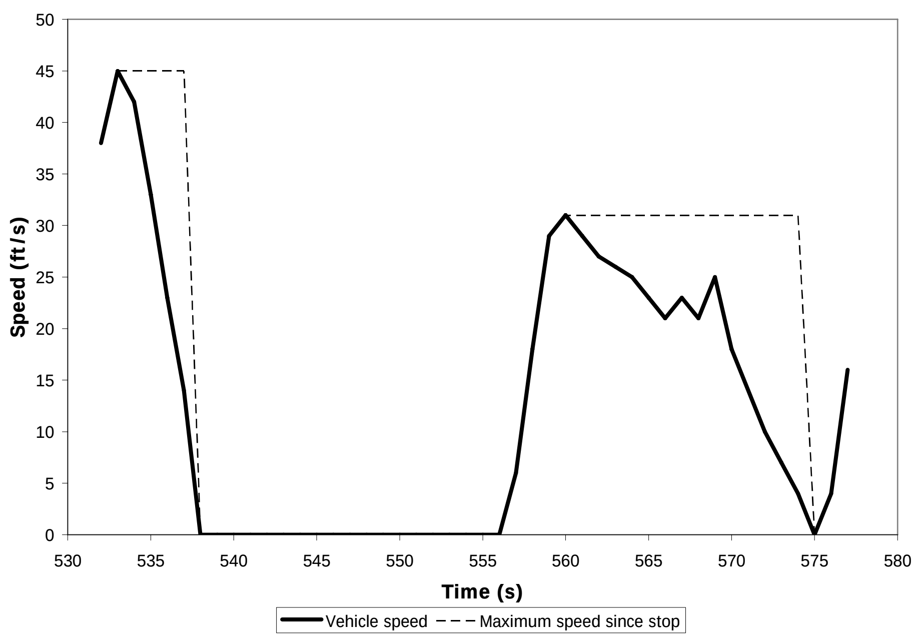


(a) Vehicle Trajectories



(b) Selected Vehicle Speed

Segment delay	Queue delay	Stop delay	Number of stops
34.64 s	33.23 s	20 s	1.81

(c) Performance Measures for Selected Vehicle

Queuing Analysis

Exhibit 24-23(a) illustrates the queue length (BOQ) per step for one lane of the signalized approach over all the time steps in the period. The 10 cycles are discernible in this figure. Also, a considerable variation in the cyclical maximum BOQ is evident.

It is important to distinguish between the percentile instantaneous BOQ and the percentile maximum BOQ per cycle. For the instantaneous BOQ, the individual observation is the BOQ on any step. So the sample size is the number of steps covered (600 in this case). For cyclical maximum BOQ, the individual observation is the maximum BOQ in any cycle, so the sample size is the number of cycles (10 in this case). The maximum BOQ in any cycle can be determined only by inspecting the plotted instantaneous values. No procedure is proposed here for automatic extraction of the maximum cyclical BOQ from the instantaneous BOQ data.

A statistical analysis showing the average BOQ, the 95th percentile BOQ (based on 2 standard deviations past the average value), and the historical maximum BOQ is presented in Exhibit 24-23(b). One important question is whether the 95% BOQ can be represented statistically on the basis of the standard deviation, assuming a normal distribution. The BOQ histogram showing the distribution of instantaneous BOQ for the 600 observations is shown in Exhibit 24-24. The appearance of this histogram does not suggest any analytical distribution; however, the relationship between the 95% BOQ and the historical maximum appears to be reasonable for this example.

Exhibit 24-23
Back of Queue Analysis by Time Step

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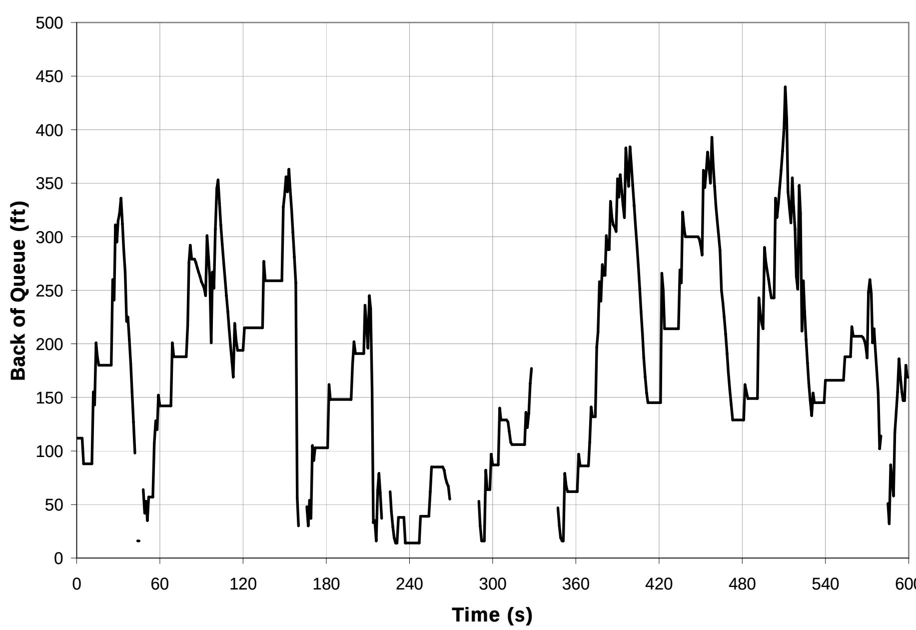

(a) Back of Queue Plot

Average queue	Standard deviation	95th percentile queue	Maximum queue
174 ft	110 ft	395 ft	440 ft

(b) Queue-Related Performance Measures

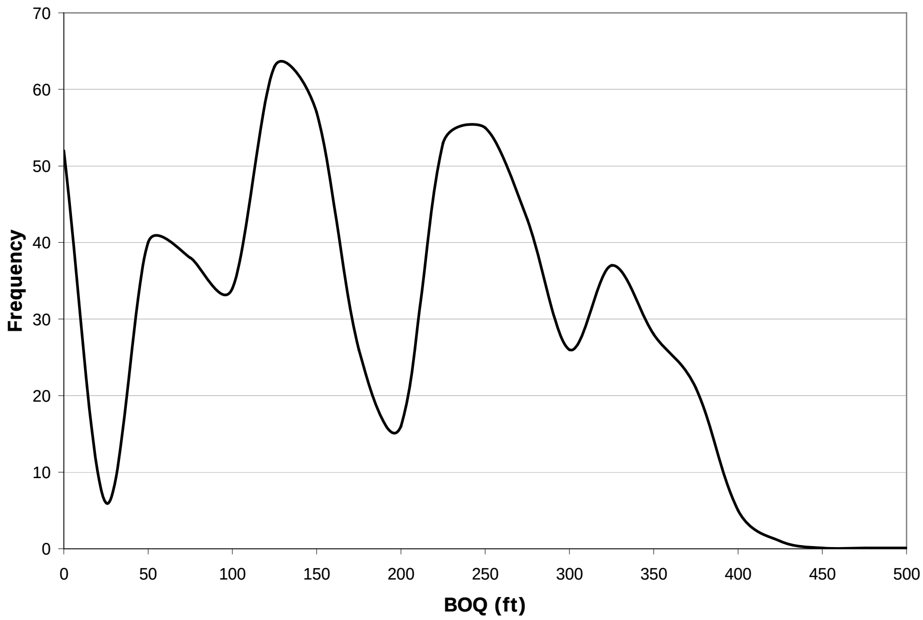

Exhibit 24-24
Back of Queue Histogram

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The queue length on an isolated approach that is approaching saturation will have a near uniform distribution (i.e., equal probability of all lengths between zero and the maximum). The standard deviation of a uniform distribution is greater than one-half of the mean, so the 95th percentile estimator (mean value plus 2 standard deviations) will be greater than the maximum value. This situation raises some doubt about the validity of basing the 95th percentile BOQ on the standard deviation, especially with cyclical queuing.

Delay Analysis for a Single Trajectory

A comparison of the accumulated delay by all definitions for the selected vehicle track indicated in Exhibit 24-20 is presented in Exhibit 24-25(a). The relationships between segment delay, queue delay, and stopped delay are evident in this figure. The segment delay begins to accumulate before the vehicle approaches the intersection because of mid-segment interactions that reduce the speed below the target speed. The queue delay begins to accumulate as the vehicle enters the queue, and the stopped delay begins to accumulate a few seconds later. The stopped delay ceases to accumulate as soon as the vehicle starts to move, but the queue delay continues to accumulate until the vehicle leaves the link.

The time step delay analysis plots shown in Exhibit 24-25(b) provide additional insight into the operation. The time step delay is close to zero as the vehicle enters the segment, indicating that the speed is close to the target speed. Small delays begin to accumulate in advance of the intersection. The accumulation becomes more rapid when the vehicle enters the queue. The periods when the vehicle is in the stopped and the queued state are also shown in this figure.

Exhibit 24-25
Accumulated Delay by Various Definitions

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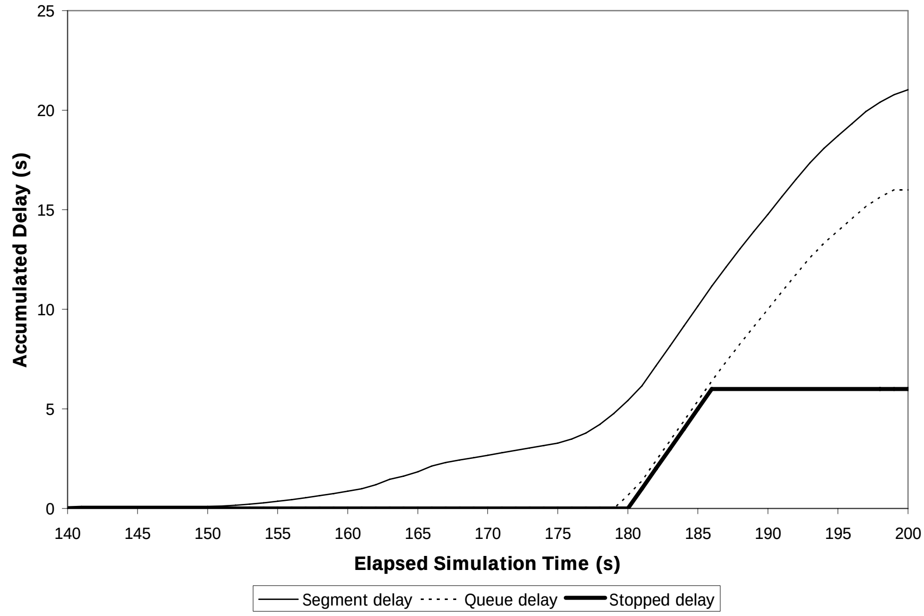
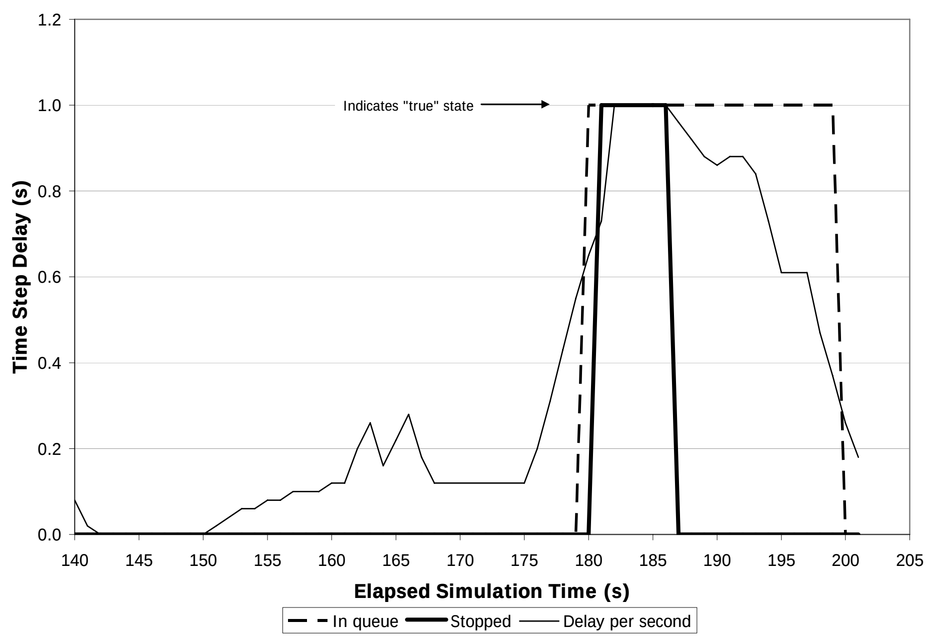


(a) Accumulated Delay



(b) Time Step Delay

It was indicated previously that the value of control delay cannot be determined by simulation in a manner that is comparable with the procedures prescribed in Chapters 18 and 31. Because this segment terminates at a signal, it is suggested that the queue delay would provide a reasonable estimate of control delay because the queue delay offers a close approximation to the delay that would be measured in the field.

Delay Analysis for All Vehicles on the Segment

The preceding example dealt with accumulated delay of a single vehicle traversing the segment. A useful delay measure requires the accumulation of delay to all vehicles traversing the segment during the period. An example is shown in Exhibit 24-26. In keeping with the recommendations offered elsewhere (*4*), only vehicles that traversed the entire link during the period are included in this analysis. Therefore, the number of vehicles analyzed (210) is lower than the number of vehicles that were actually on the link during the period (286).

Exhibit 24-26
Delay Analysis for All Vehicles on a Segment

	Segment Delay (s)	Queue Delay (s)	Stop Delay (s)	No. of Stops
Lane 1	3,128	2,562	1,957	95.4
Lane 2	3,400	2,793	2,047	96.2
Total	6,529	5,355	4,004	191.6
Average per vehicle	31.09	25.50	19.07	0.91

Analysis of a Freeway Segment

A performance analysis of the freeway merge area originally shown in Exhibit 24-9 is presented here. A single vehicle is selected from the trajectory plot and its trajectory is analyzed. The results are shown in Exhibit 24-27. The analysis produced segment delay and queue delay. This segment was very congested, as indicated by the trajectory plot. No stopped delay was produced because the vehicle never actually came to a stop (i.e., its speed stayed above 5 mi/h).

Exhibit 24-27
Longitudinal Analysis of Delay for a Selected Vehicle in a Merge Area

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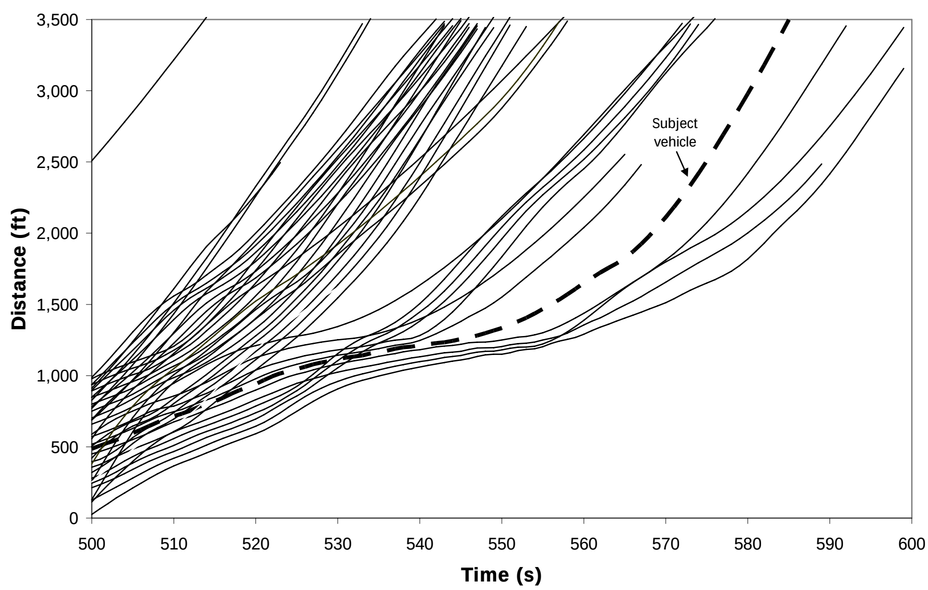

(a) Vehicle Trajectories

Segment delay	Queue delay	Stopped delay
39.58 s	37.01 s	0 s

(b) Delay-Related Performance Measures for Subject Vehicle

A spatial analysis of the entire segment can also be performed to produce the following measures by lane:

- Average density over the segment,
- Percent slow vehicles (i.e., traveling at less than two-thirds the target speed),
- Percent queued vehicles,
- Average queue length (measured from front of queue to BOQ),
- Average BOQ position,
- Maximum BOQ position, and
- Percent of time steps when the queue overflowed the segment.

The results are presented in tabular form in Exhibit 24-28. The values are presented by lane and are combined for Lanes 1 and 2 for compatibility with the HCM definition of merge area density.

Exhibit 24-28
Example Spatial Analysis by Lane

	Lane 1	**Lane 2**	**Lane 3**	**Acceleration Lane**
Average density (veh/mi/ln)	73.4	51.0	43.6	9.9
Percent slow vehicles (%)	88.4	68.5	41.5	65.7
Percent queued vehicles (%)	63.4	22.0	2.4	26.7
Average queue length (ft)	600	215	15	40
Average back of queue (ft)	1,471	1,119	135	562
Maximum back of queue (ft)	1,497	1,497	1,492	1,474
Percent overflow	66.1	29.6	0.5	0.17

Note: Average Lane 1 and Lane 2 density is 62.2 veh/mi/ln.

2. REFERENCES

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2. Van As, S. C., and A. Van Niekerk. The Operational Analysis of Two-Lane Rural Highways. Presented at 23rd Annual Southern African Transport Conference, Pretoria, South Africa, July 2004.
3. Catbagan, J. L., and H. Nakamura. Probability-Based Follower Identification in Two-Lane Highways. Presented at 88th Annual Meeting of the Transportation Research Board, Washington, D.C., 2009.
4. Dowling, R. *Traffic Analysis Toolbox Volume VI: Definition, Interpretation, and Calculation of Traffic Analysis Tools Measures of Effectiveness*. Federal Highway Administration, Washington, D.C., 2007.

Many of these references can be found in the Technical Reference Library in Volume 4.

CHAPTER 25
FREEWAY FACILITIES: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

SCOPE OF CHAPTER

The freeway facility analytical methodology is described in Chapter 10. The computations contained in the methodology are detailed in this supplemental chapter. The documentation is closely tied to the computational engine for Chapter 10: FREEVAL-2010.

The FREEVAL (FREeway EVALuation) tool was initially developed for the 2000 edition of the *Highway Capacity Manual* (HCM) (*1, 2*) and has been updated to reflect methodological changes in HCM 2010. All definitions and labels of variables and subroutines in this chapter are consistent with the computational code in FREEVAL. The Technical Reference Library in Volume 4 contains a FREEVAL user guide, which provides more details on how to use the computational engine.

Section 2 of this chapter presents a glossary of all relevant variables used in the procedure and the computational engine. Section 3 describes the overall procedure, and Sections 4 and 5 provide additional detail for the undersaturated and oversaturated flow regimes, respectively. Section 6 describes the directional facility module that aggregates performance measures for the overall facility. The chapter concludes with the formal documentation of the computational engine in Section 7 in the form of flowcharts and linkage lists.

LIMITATIONS

The completeness of the analysis will be limited if freeway segment cells in the first time interval, the last time interval, and the first freeway segment do not have demand-to-capacity ratios of 1.00 or less. The methodology can handle congestion in the first interval properly, although it will not quantify any congestion that could have occurred before the analysis. To ensure complete quantification of the effects of congestion, it is recommended that the analysis contain an initial undersaturated time interval. If all freeway segments in the last time interval do not exhibit demand-to-capacity ratios less than 1.00, congestion continues beyond the last time interval, and additional time intervals should be added. This fact will be noted as a difference between the vehicle miles of travel desired at the end of the analysis (demand flow) and the corresponding vehicle miles of travel flow generated (volume served). If queues extend upstream of the first segment, the analysis will not account for the congestion outside the freeway facility but will store the vehicles vertically until the congestion clears the first segment. The same process is followed for queues on on-ramp segments.

The methodology for oversaturated flow conditions described in this chapter is based on concepts of traffic flow theory and assumes a linear speed–flow relationship for densities greater than 45 passenger cars per mile per lane (pc/mi/ln). This relationship has not been extensively calibrated for field observations on U.S. freeways, and analysts should therefore perform their own validation from local data to obtain additional confidence in the results of this procedure. For an example of a validation exercise for this methodology, the reader is referred elsewhere (*3*).

The procedure described here becomes extremely complex when the queue from a downstream bottleneck extends into an upstream bottleneck, causing a queue collision. When such cases arise, the reliability of the methodology is questionable, and the user is cautioned about the validity of the results. For heavily congested directional freeway facilities with interacting bottleneck queues, a traffic simulation model might be more applicable. Noninteracting bottlenecks are addressed by the methodology.

The procedure focuses on analyzing a directional series of freeway segments. It describes the performance of a facility but falls short of addressing the broader transportation network. The analyst is cautioned that severe congestion on a freeway—especially freeway on-ramps—is likely to have an impact on the adjacent surface street network. Similarly, the procedure is limited in its ability to predict the impacts of an oversaturated off-ramp and the associated queues that may spill back onto the freeway. Alternative tools are suitable to evaluate these impacts.

2. GLOSSARY OF VARIABLE DEFINITIONS

This glossary defines internal variables used exclusively in the freeway facilities methodology. The variables are consistent with those used in the FREEVAL-2010 computational engine for the freeway facilities methodology. The glossary of variables covers six parts: global variables, segment variables, node variables, on-ramp variables, off-ramp variables, and facilitywide variables. Segment variables represent conditions on segments. Node variables denote flows across a node connecting two segments. On-ramp and off-ramp variables are variables that correspond to flow on ramps. Facilitywide variables pertain to aggregate traffic performance over the entire facility. In addition to these spatial categories, there are temporal divisions that represent characteristics over a time step or a time interval. The first dimension associated with each variable specifies whether the variable refers to segment or node characteristics. The labeling scheme for nodes and segments is such that segment i is immediately downstream of node i. The distinction of nodes and segments is used primarily in the oversaturated flow regime as discussed in Section 5 of this chapter.

Thus, there is always one more node than the number of segments on a facility. The second and third dimensions denote a time step t and a time interval p. Facility variables are estimates of the average performance over the length of the facility. The units of flow are in vehicles per time step. The selection of the time step size is discussed later in this chapter.

The variable symbols used internally by FREEVAL and replicated in this chapter frequently differ from the symbols used elsewhere in the HCM, particularly in Chapter 10, Freeway Facilities. For example, the HCM uses D_{jam} to represent jam density, whereas FREEVAL and this chapter use KJ.

GLOBAL VARIABLES

- i—index to segment or node number: $i = 1, 2, \ldots, NS$ (for segments) and $i = 1, 2, \ldots, NS + 1$ (for nodes).
- KC—ideal density at capacity [vehicles per mile per lane (veh/mi/ln)]. The density at capacity is 45 pc/mi/ln, which must be converted to veh/mi/ln by using the heavy-vehicle adjustment factor (f_{HV}) described in Chapter 11, Basic Freeway Segments.
- KJ—facilitywide jam density (veh/mi/ln).
- NS—number of segments on the facility.
- P—number of time intervals in the analysis period.
- p—time interval number: $p = 1, 2, \ldots, P$.
- S—number of time steps in a time interval (integer).
- t—time step number in a single interval: $t = 1, 2, \ldots, S$.
- T—number of time steps in 1 h (integer).

SEGMENT VARIABLES

- *ED*(*i*, *p*)—expected demand that would arrive at segment *i* on the basis of upstream conditions over time interval *p*. The upstream queuing effects include the metering of traffic from an upstream queue but not the spillback of vehicles from a downstream queue.
- *K*(*i*, *p*)—average traffic density of segment *i* over time interval *p*, as estimated by the oversaturated procedure.
- *KB*(*i*, *p*)—background density: segment *i* density (veh/mi/ln) over time interval *p* assuming there is no queuing on the segment. This density is calculated by using the expected demand on the segment in the corresponding undersaturated procedure in Chapters 11, 12, and 13.
- *KQ*(*i*, *t*, *p*)—queue density: vehicle density in the queue on segment *i* during time step *t* in time interval *p*. Queue density is calculated on the basis of a linear density–flow relationship in the congested regime.
- *L*(*i*)—length of segment *i* (mi).
- *N*(*i*, *p*)—number of lanes on segment *i* in time interval *p*. It could vary by time interval if a temporary lane closure is in effect.
- *NV*(*i*, *t*, *p*)—number of vehicles present on segment *i* at the end of time step *t* during time interval *p*. The number of vehicles is initially based on the calculations of Chapters 11, 12, and 13, but, as queues grow and dissipate, input–output analysis updates these values in each time step.
- *Q*(*i*, *t*, *p*)—total queue length on segment *i* at the end of time step *t* in time interval *p* (ft).
- *SC*(*i*, *p*)—segment capacity: maximum number of vehicles that can pass through segment *i* in time interval *p* based strictly on traffic and geometric properties. These capacities are calculated by using Chapters 11, 12, and 13.
- *SD*(*i*, *p*)—segment demand: desired flow rate through segment *i* including on- and off-ramp demands in time interval *p* (veh). This segment demand is calculated without any capacity constraints.
- *SF*(*i*, *t*, *p*)—segment flow out of segment *i* during time step *t* in time interval *p* (veh).
- *U*(*i*, p)—average space mean speed over the length of segment *i* during time interval *p* (mi/h).
- *UV*(*i*, *t*, *p*)—unserved vehicles: the additional number of vehicles stored on segment *i* at the end of time step *t* in time interval *p* due to a downstream bottleneck.
- *WS*(*i*, *p*)—wave speed: speed at which a front-clearing queue shock wave travels through segment *i* during time interval *p* (ft/s).
- *WTT*(*i*, *p*)—wave travel time: time taken by the shock wave traveling at wave speed *WS* to travel from the downstream end of segment *i* to the upstream end of the segment during time interval *p*, in time steps.

NODE VARIABLES

- *MF*(*i*, *t*, *p*)—actual mainline flow rate that can cross node *i* during time step *t* in time interval *p*.
- *MI*(*i*, *t*, *p*)—maximum mainline input: maximum flow desiring to enter node *i* during time step *t* in time interval *p*, based on flows from all upstream segments, taking into account all geometric and traffic constraints upstream of the node, including queues accumulated from previous time intervals.
- *MO1*(*i*, *t*, *p*)—maximum Mainline Output 1: maximum allowable mainline flow rate across node *i* during time step *t* in time interval *p*, limited by the flow from an on-ramp at node *i*.
- *MO2*(*i*, *t*, *p*)—maximum Mainline Output 2: maximum allowable mainline flow rate across node *i* during time step *t* in time interval *p*, limited by available storage on segment *i* due to a downstream queue.
- *MO3*(*i*, *t*, *p*)—maximum Mainline Output 3: maximum allowable mainline flow rate across node *i* during time step *t* in time interval *p*, limited by the presence of queued vehicles at the upstream end of segment *i* while the queue clears from the downstream end of segment *i*.

ON-RAMP VARIABLES

- *ONRC*(*i*, *p*)—geometric carrying capacity of on-ramp at node *i* roadway during time interval *p*.
- *ONRD*(*i*, *p*)—demand flow rate for on-ramp at node *i* in time interval *p*.
- *ONRF*(*i*, *t*, *p*)—actual ramp flow rate that can cross on-ramp node *i* during time step *t* in time interval *p*; takes into account control constraints (e.g., ramp meters).
- *ONRI*(*i*, *t*, *p*)—input flow rate desiring to enter the merge point at on-ramp *i* during time step *t* in time interval *p*, based on current ramp demand and ramp queues accumulated from previous time intervals.
- *ONRO*(*i*, *t*, *p*)—maximum output flow rate that can enter the merge point from on-ramp *i* during time step *t* in time interval *p*; constrained by Lane 1 (shoulder lane) flow on segment *i* and the segment *i* capacity or by a queue spillback filling the mainline segment from a bottleneck further downstream, whichever governs.
- *ONRQ*(*i*, *t*, *p*)—unmet demand that is stored as a queue on the on-ramp roadway at node *i* during time step *t* in time interval *p* (veh).
- *ONRQL*(*i*, *t*, *p*)—queue length on on-ramp *i* at the end of time step *t* in time interval *p*.
- *RM*(*i*, *p*)—maximum allowable rate of an on-ramp meter at the on-ramp at node *i* during time interval *p* (veh/h).

OFF-RAMP VARIABLES

- *DEF*(*i*, *t*, *p*)—deficit: unmet demand from a previous time interval *p* that flows past node *i* during time step *t*; used in off-ramp flow calculations downstream of a bottleneck.
- *OFRD*(*i*, *p*)—desired off-ramp demand flow exiting at off-ramp *i* during time interval *p*.
- *OFRF*(*i*, *t*, *p*)—actual flow that can exit at off-ramp *i* during time step *t* in time interval *p*.

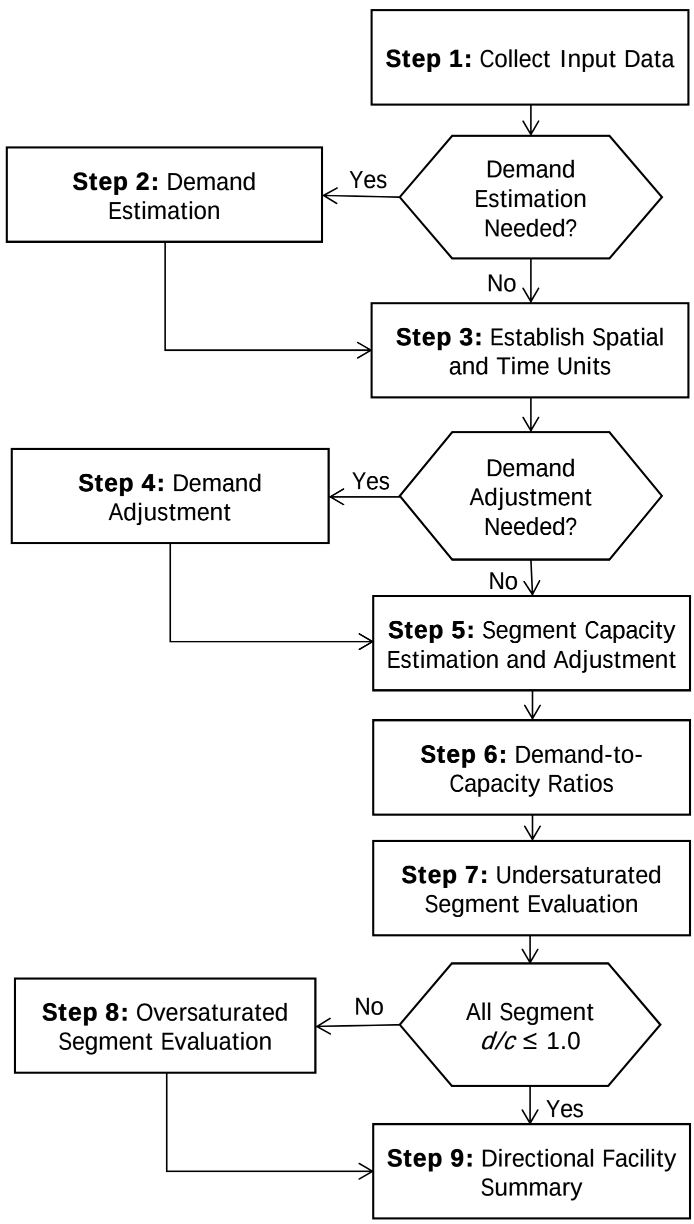
FACILITYWIDE VARIABLES

- *K*(*NS*, *P*)—average vehicle density over the entire facility during the entire analysis period *P*.
- *K*(*NS*, *p*)—average vehicle density over the entire facility during time interval *p*.
- *SMS*(*NS*, *P*)—average analysis period facility speed: average space mean speed over the entire facility during the entire analysis period *P*.
- *SMS*(*NS*, *p*)—average time interval facility speed: average space mean speed over the entire facility during time interval *p*.

3. OVERALL PROCEDURE DESCRIPTION

The procedure is described according to the nine-step process shown in Exhibit 25-1. The methodology is presented in a logical step-by-step process flow. Readers interested in technical details on how this methodology is implemented in the FREEVAL computational engine are referred to Section 7 of this chapter.

Exhibit 25-1
Overall Procedure Layout



STEP 1: COLLECT INPUT DATA

The first step in the methodology is to gather all geometric and traffic data. The most basic data are required for sizing the analysis. These basic data are listed below.

- *Number of time intervals:* the number of time intervals is input to size the analysis with the correct time dimension. There is no practical limit on the number of time intervals, although the current computational engine implementation is limited to 24 intervals.
- *Time interval duration:* the time interval duration is fixed to 15 min. The methodology assumes that there is instantaneous travel time between segments when demands are computed on segments. In other words, there is no demand shock wave at any point where the demand changes (i.e., when a new time interval begins). For this assumption to be reasonable, the uncongested travel time of the freeway facility being analyzed (which is directly related to its length) should not be longer than the duration of the time intervals being used.
- *Time step duration:* once oversaturation begins, the procedure moves to time steps. The analysis interval is reduced from a 15-min time interval to 1-min time steps to be able to track queuing effects in sufficient detail.
- *Number of segments:* the number of segments in the facility must be determined from the process given in Chapter 10, Freeway Facilities.
- *Jam density:* the systemwide jam density is required for oversaturated analysis. The default value is 190 pc/mi/ln, but jam density can be calibrated from user input.

The geometric, traffic, and demand data required for a freeway facility analysis are discussed in Chapter 10.

STEP 2: DEMAND ESTIMATION

The demand estimation module is needed when the methodology uses actual freeway counts and actual demand flows are unknown. If demand flows are known or can be projected, those values can be used directly. FREEVAL-2010 assumes that demand volumes are known; if they are not, the analyst should follow the discussion below to develop a demand volume matrix for the facility to input into the computational engine.

The demand estimation module is designed to convert the input set of freeway 15-min traffic counts into a set of freeway 15-min traffic demands. Freeway demand is defined as the number of vehicles that desire to use the freeway in a given 15-min time interval. This demand may not be represented by the 15-min count because of upstream freeway congestion within the freeway facility.

To estimate demands, the analyst sums the freeway entrance demands along the entire freeway facility (including the freeway mainline entrance) and compares it with the sum of the freeway exit counts along the entire freeway facility (including the freeway mainline exit) for each time interval. The ratio of the freeway entrance demands to the freeway exit counts is calculated for each time interval and is referred to as the *time interval scale factor*. Theoretically, the scale factor should approach 1.00 when the freeway exit counts are, in fact, freeway exit demands.

Scale factors greater than 1.00 indicate increasing congestion within the freeway facility (and storing of vehicles on the freeway). Here, the exit traffic counts underestimate the actual freeway exit demands. Scale factors less than 1.00 indicate decreasing congestion within the freeway facility (and release of stored vehicles on the freeway). Here, the exit traffic counts overestimate actual freeway exit demands. To obtain an estimate of freeway exit demand, each freeway exit count in the time interval is multiplied by the time interval scale factor.

*To obtain an estimate of freeway exit **demand**, each freeway exit **count** in the time interval is multiplied by the time interval scale factor.*

The accuracy of this procedure depends primarily on the quality of the freeway traffic counts and to a lesser extent on the length of the freeway facility. With the use of 15-min time intervals, freeway facility lengths up to 9 to 12 mi should not introduce significant errors in the procedure. The calculated scale factor pattern over the study period offers a means of checking the quality of the traffic count data. For example, if there is no congestion over the entire time–space domain, then there should be no pattern in the calculated 15-min scale factors, and they all should be within the range of 0.95 to 1.05.

If there is congestion within the time–space domain, then there should be a pattern in the calculated 15-min scale factors. During the early time intervals with no congestion, the scale factors are expected to approach 1.00 and to be within the range of 0.95 to 1.05. As congestion begins to occur and to increase over time, the scale factors are expected to exceed 1.00 and to be within the range of 1.00 to 1.10. When the extent of congestion reaches its highest level, the scale factor is expected to approach 1.00 and to be within the range of 0.95 to 1.05.

As the level of congestion recedes, the scale factor is expected to be less than 1.00 and to be within the range of 0.90 to 1.00. If the final time intervals exhibit no congestion over the complete time–space domain, then there should be no pattern in the calculated 15-min scale factors, and they all should be within the range of 0.95 to 1.05. Once the freeway entrance and exit demands are estimated by using the scale factors, the traffic demands for each freeway section in each time interval can be determined.

STEP 3: ESTABLISH SPATIAL AND TIME UNITS

The procedure analyzes a freeway in spatial units called segments, which are defined in Chapters 11, 12, and 13. The division of a freeway facility into segments is described in Chapter 10. Time units are described later in this chapter.

STEP 4: DEMAND ADJUSTMENT

Driver responses such as spatial, temporal, and modal shifts caused by traffic management strategies are not automatically incorporated in the methodology. On viewing the facility traffic performance results, the analyst can modify the demand input manually to simulate the effect of user demand responses or traffic growth effects. The accuracy of the results depends on the accuracy of the estimation of the user demand responses. Ramp-metering strategies are evaluated through adjusting the on-ramp roadway capacity, and this application is described in the segment capacity adjustment module.

The computational engine further provides origin–destination demand adjustment factors to adjust demand flows automatically by a multiplicative growth or shrinkage factor. For example, an origin demand adjustment factor of 1.15 will increase on-ramp demand by 15%. These factors can be applied to any segment in any time period. Separate origin and destination demand adjustment factors are provided to adjust on-ramp and off-ramp demand flows, respectively. By specifying a common factor to all segments and time periods, the analyst can test uniform overall growth in traffic demand.

STEP 5: SEGMENT CAPACITY ESTIMATION AND ADJUSTMENT

Actual field-observed capacities at bottlenecks should be obtained when practical and should replace estimated capacities.

Segment capacity estimates are determined from Chapters 11, 12, and 13 for basic freeway segments, weaving segments, and ramp segments, respectively. All capacities are expressed in vehicles per hour. All estimates of segment capacity should be carefully reviewed and compared with local knowledge and available traffic information for the study site. The capacity value used for bottleneck segments has the greatest effect on predicted freeway traffic performance. Actual field-observed capacities at bottlenecks should be obtained when practical and should replace estimated capacities.

On-ramp and off-ramp roadway capacities are also determined in this capacity module. On-ramp demands may exceed on-ramp capacities and limit the traffic demand entering the freeway. Off-ramp demands may exceed off-ramp capacities and cause congestion on the freeway, although this particular effect is not accounted for in the methodology and thus requires the use of alternative tools. The relationships of demand and capacity for each on-ramp and off-ramp, as well as for each freeway segment, are addressed later in the demand-to-capacity ratio module.

Again, unlike the analyses in the basic freeway segment, weaving, and ramp chapters, all analyses in this chapter measure capacity by using vehicles, rather than passenger cars, as the units.

The effect of a predetermined ramp-metering plan can be evaluated in this methodology by modifying the ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to the desired metering rate specified. This feature not only allows for evaluating the prespecified ramp-metering plan but also permits the user to experiment to obtain an improved ramp-metering plan.

Freeway design improvements can be evaluated within this methodology by modifying the design features of any segment or segments of the freeway facility, as described later in this chapter.

Reduced-capacity situations can also be investigated. The capacity in any cell of the time–space domain can be reduced to represent incident situations such as construction and maintenance activities, adverse weather, traffic accidents, and vehicle breakdowns. Similarly, capacity can be increased to match field measurements. When adjusted capacity situations are analyzed, it is important to use an alternative speed–flow relationship. The predicted speed for the alternative speed–flow model can be computed by using Equation 25-1. The

relationship ensures a constant ideal density of 45 pc/mi/ln at capacity as indicated in Chapter 11, Basic Freeway Segments.

Equation 25-1

$$S = FFS + \left[1 - e^{\ln\left(FFS + 1 - \frac{C \times CAF}{45}\right)\frac{v_p}{C \times CAF}} \right]$$

where

S = segment speed (mi/h),

FFS = segment free-flow speed (mi/h),

C = original segment capacity (pc/h/ln),

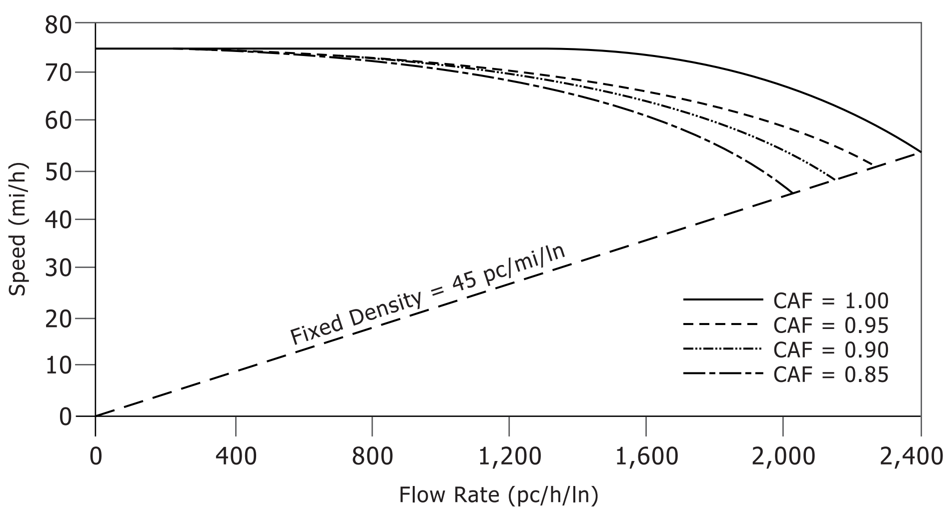
CAF = capacity adjustment factor (CAF = 1.0, use speed estimation procedures in Chapters 11, 12, and 13), and

v_p = segment flow rate (pc/h/ln).

Note that when v_p approaches zero in Equation 25-1, S approaches the free-flow speed. Similarly, when $v_p = C \times CAF$, S approaches the speed at capacity. Exhibit 25-2 shows speed–flow plots for capacity adjustment factors of 100%, 95%, 90%, and 85% of the original capacity.

Exhibit 25-2
Alternative Speed–Flow Curves for Indicated Capacity Adjustment Factors

LIVE GRAPH
Click here to view



Note: Free-flow speed = 75 mi/h (base conditions); *CAF* = capacity adjustment factor (proportion of available capacity).

STEP 6: DEMAND-TO-CAPACITY RATIOS

Once all freeway segment cells have been analyzed, demand-to-capacity ratios are modified into volume-to-capacity ratios for later use in calculating freeway traffic performance measures. As stated earlier, if all freeway segment cells are undersaturated (demands less than capacities), the volume-to-capacity ratios are identical to the demand-to-capacity ratios, and the analysis is simple. However, if demand exceeds capacity in one or more of the freeway segment cells, oversaturated flow conditions will occur, and the time step analysis procedure is invoked.

Until oversaturated conditions are encountered, segments are analyzed by using the undersaturated segment measure-of-effectiveness (MOE) module. All

subsequent time intervals, however, are analyzed by using the oversaturated segment MOE module until all queues have cleared from the facility.

STEP 7: UNDERSATURATED SEGMENT EVALUATION

Segments are analyzed by using the Chapter 11, 12, and 13 methodologies for all time periods when the demand-to-capacity ratio is less than or equal to 1.0, *and* when there are no residual queues from an earlier period of congestion. The undersaturated segment evaluation is discussed in more detail in Section 4 of this chapter.

STEP 8: OVERSATURATED SEGMENT EVALUATION

The oversaturated segment evaluation is used once the demand on a segment exceeds its capacity ($v_d/c > 1.0$) in a time period. The oversaturated procedures are used as long as v_d/c remains greater than 1.0 and as long as a queue remains in the segment (even after v_d/c has dropped back below 1.0). Depending on the extent of the congestion, a queue is likely to spill over into adjacent upstream segments. If that occurs, the oversaturated evaluation is also used for these upstream segments, even though the v_d/c ratios for these segments may be less than 1.0. The oversaturated segment evaluation is discussed in more detail in Section 5 of this chapter.

STEP 9: DIRECTIONAL FACILITY SUMMARY

In the final analysis step, the calculated segment performance measures (e.g., average speed, density, travel time) are aggregated over the entire directional facility. The directional facility summary is discussed in more detail in Section 6 of this chapter.

4. UNDERSATURATED SEGMENT EVALUATION

This module begins with the first segment in the first time interval. For each cell, the flow (or volume) is equal to demand, the volume-to-capacity ratio is equal to the demand-to-capacity ratio, and undersaturated flow conditions prevail. Performance measures for the first segment during the first time interval are calculated by using the procedures for the corresponding segment type in Chapters 11, 12, and 13.

The analysis continues to the next downstream freeway segment in the same time interval, and the performance measures are calculated. The process is continued until the last downstream freeway segment cell in this time interval has been analyzed. For each cell, the volume-to-capacity ratio and performance measures are calculated for each freeway segment in the first time interval. The analysis continues in the second time interval beginning at the furthest upstream freeway segment and moving downstream until all freeway segments in that time interval have been analyzed. This pattern continues for the third time interval, fourth time interval, and so on until the methodology encounters a time interval that contains one or more segments with a demand-to-capacity ratio greater than 1.00 or when the last segment in the last time interval is analyzed. If no oversaturated segments are encountered, the segment performance measures are taken directly from Chapters 11, 12, and 13, and the facility performance measures are calculated as described in Section 6 of this chapter.

When the analysis moves from isolated segments to a facility, an additional constraint is necessary that controls the relative speed between two segments. To limit the speeds downstream of a segment experiencing a low average speed, a maximum achievable speed is imposed on the downstream segments. This maximum speed is based on acceleration characteristics reported elsewhere (*4*) and is shown in Equation 25-2.

On the facility level, a speed constraint is introduced that limits the maximum achievable speed downstream of a segment experiencing a low average speed.

Equation 25-2

$$V_{max} = FFS - (FFS - V_{prev})e^{-0.00162L}$$

where

V_{max} = maximum achievable segment speed (mi/h),

FFS = segment free-flow speed (mi/h),

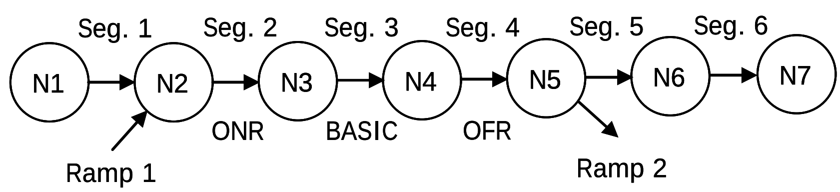
V_{prev} = average speed on immediate upstream segment (mi/h), and

L = distance from midpoints of the upstream segment and the subject segment (ft).

5. OVERSATURATED SEGMENT EVALUATION

Oversaturated flow conditions occur when the demand on one or more freeway segment cells exceeds its capacity. Once oversaturation is encountered, the methodology changes its temporal and spatial units of analysis. The spatial units become nodes and segments, and the temporal unit moves from a time interval to smaller time steps. A node is defined as the junction of two segments. There is always one more node than segment, with a node added at the beginning and end of each segment. The numbering of nodes and segments begins at the upstream end and moves to the downstream end, with the segment upstream of node i numbered segment $i - 1$ and the downstream segment numbered i, as shown in Exhibit 25-3. The intermediate segments and node numbers represent the division of the section between Ramps 1 and 2 into three segments numbered 2 (ONR), 3 (BASIC), and 4 (OFR). The oversaturated analysis moves from the first node to each downstream node in the same time step. After completion of a time step, the same nodal analysis is performed for subsequent time steps.

Exhibit 25-3
Node–Segment Representation of a Directional Freeway Facility



The oversaturated analysis focuses on the computation of segment average flows and densities in each time interval. These parameters are later aggregated to produce facilitywide estimates. Two key inputs into the flow estimation procedures are the time step duration for flow updates and a flow–density function. They are described in the next subsections.

PROCEDURE PARAMETERS

Time Step Duration

Segment flows are calculated in each time step and are used to calculate the number of vehicles on each segment at the end of every time step. The number of vehicles on each segment is used to track queue accumulation and discharge and to calculate the average segment density.

To provide accurate estimates of flows in oversaturated conditions, the time intervals are divided into smaller time steps. The conversion from time intervals to time steps occurs during the first oversaturated time interval and remains until the end of the analysis. The transition to time steps is essential because, at certain points in the methodology, future performance estimates are made on the basis of the past value of a variable. Time steps correspond to the following lengths in Exhibit 25-4.

Exhibit 25-4
Recommended Time Step Duration for Oversaturated Analysis

Shortest segment length (ft)	≤300	600	1,000	1,300	≥1,500
Time step duration (s)	15	25	40	60	60

The FREEVAL-2010 computational engine assumes a time step of 15 s for the oversaturated flow computations, which is adequate for most facilities with a minimum segment length greater than 300 ft. For shorter segments, two problem situations may arise. The first situation occurs when segments are short and the rate of queue growth is rapid. Under these conditions, a short segment may be completely undersaturated in one time step and completely queued in another. The methodology may store more vehicles in this segment during a time step than space allows. Fortunately, the next time step compensates for this error, and the procedure continues to track queues and store vehicles accurately after this correction.

The oversaturated methodology as implemented in FREEVAL-2010 assumes a time step of 15 s, which is adequate for segment lengths greater than 300 ft.

The second situation in which small time steps are important occurs when two queues interact. There is a temporary inaccuracy due to the maximum output of a segment changing, thus causing the estimation of available storage to be slightly in error. This situation results in the storage of too many vehicles on a particular segment. This supersaturation is temporary and is compensated for in the next time step. Inadequate time step size will result in erroneous estimation of queue lengths and may affect other performance measures as well. Regardless, if interacting queues occur, the results should be viewed with extreme caution.

Flow–Density Relationship

Analysis of freeway segments depends on the relationships between segment speed, flow, and density. Chapter 11, Basic Freeway Segments, defines a relationship between these variables and the calculation of performance measures in the undersaturated regime. The freeway facilities methodology presented here uses the same relationships for undersaturated segments. In other words, when a segment is undersaturated the computations of this methodology are identical to the results obtained from Chapters 11, 12, and 13 for basic freeway segments, weaving segments, and ramp segments, respectively.

The calculations for oversaturated segments assume a simplified linear flow–density diagram in the congested region. Exhibit 25-5 shows this flow–density diagram for a segment having a free-flow speed of 75 mi/h. For other free-flow speeds, the corresponding capacities in Chapters 11, 12, and 13 should be used.

The oversaturated regime curve in Exhibit 25-5 is constructed from a user-specified jam density (default is 190 pc/mi/ln) and the known value of capacity, defined as the flow at a density of 45 pc/mi/ln. The flow–density relationship is assumed to be linear between these two points. The slope of the resulting line describes the speed of the shock wave at which queues grow and dissipate as discussed further below. The speed in a congested segment is obtained from the prevailing density in the segment, read along the linear flow–density relationship. Details on the theory of kinematic waves in highway traffic are given elsewhere (*5*–*7*).

Exhibit 25-5
Segment Flow–Density Function

LIVE GRAPH
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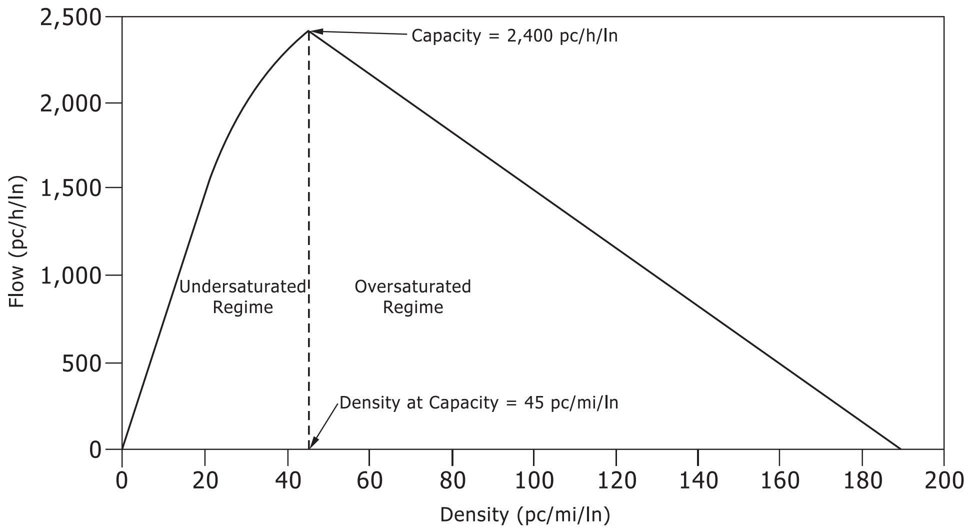


Note: Assumed *FFS* = 75 mi/h.

FLOW ESTIMATION

The oversaturated portion of the methodology is detailed as a flowchart in Exhibit 25-6. The flowchart is divided into several parts, which are discussed in this section. Within each subsection, computations are detailed and labeled according to each step of the flowchart.

The procedure first calculates a number of flow variables starting at the first node during the first time step of oversaturation, followed by each downstream node and segment in the same time step. After all computations in the first time step are completed, calculations are performed at each node and segment during subsequent time steps for all remaining time intervals until the analysis is completed.

Exhibit 25-6
Oversaturated Analysis Procedure

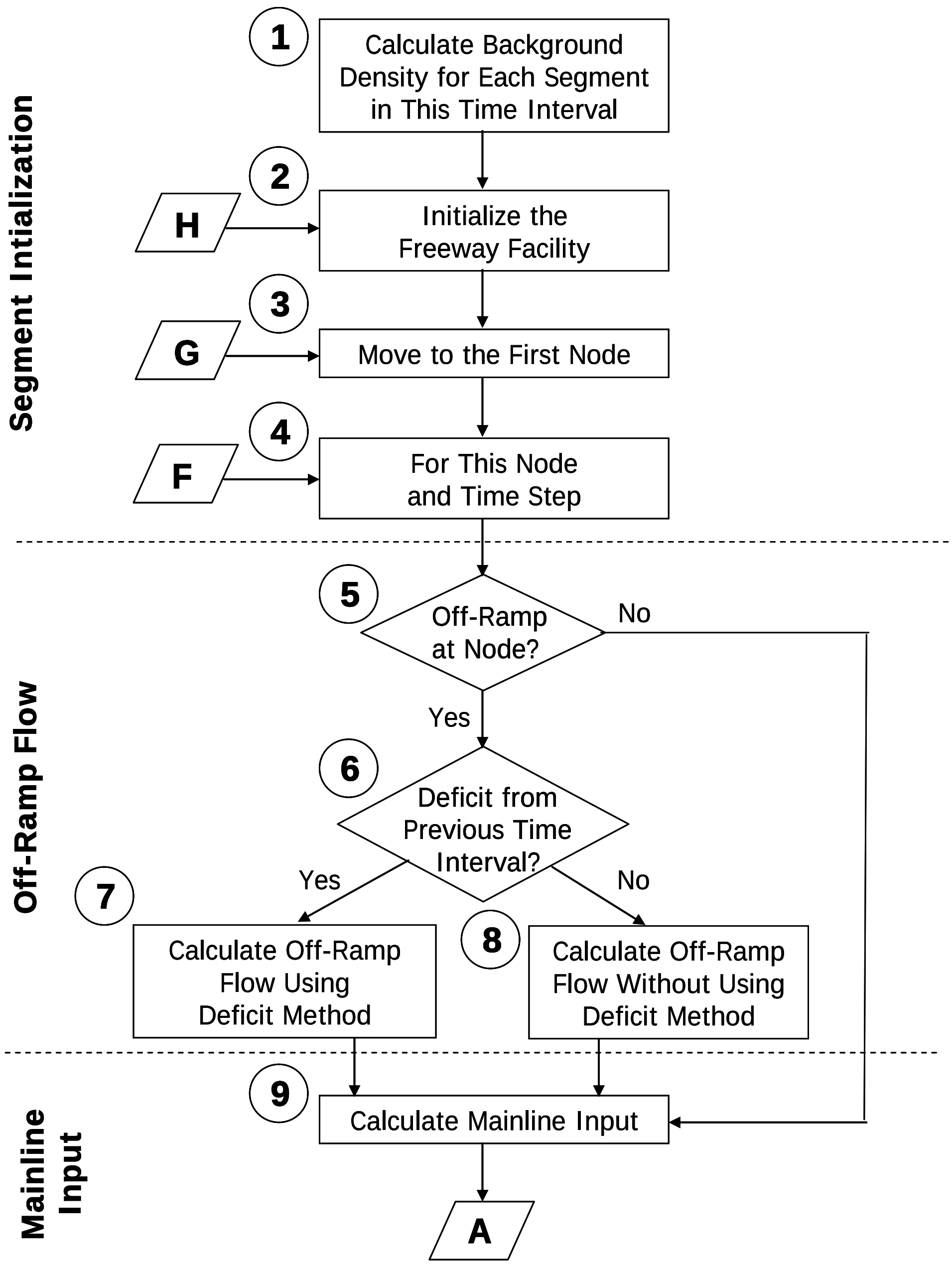


Exhibit 25-6 (cont'd.)
Oversaturated Analysis Procedure

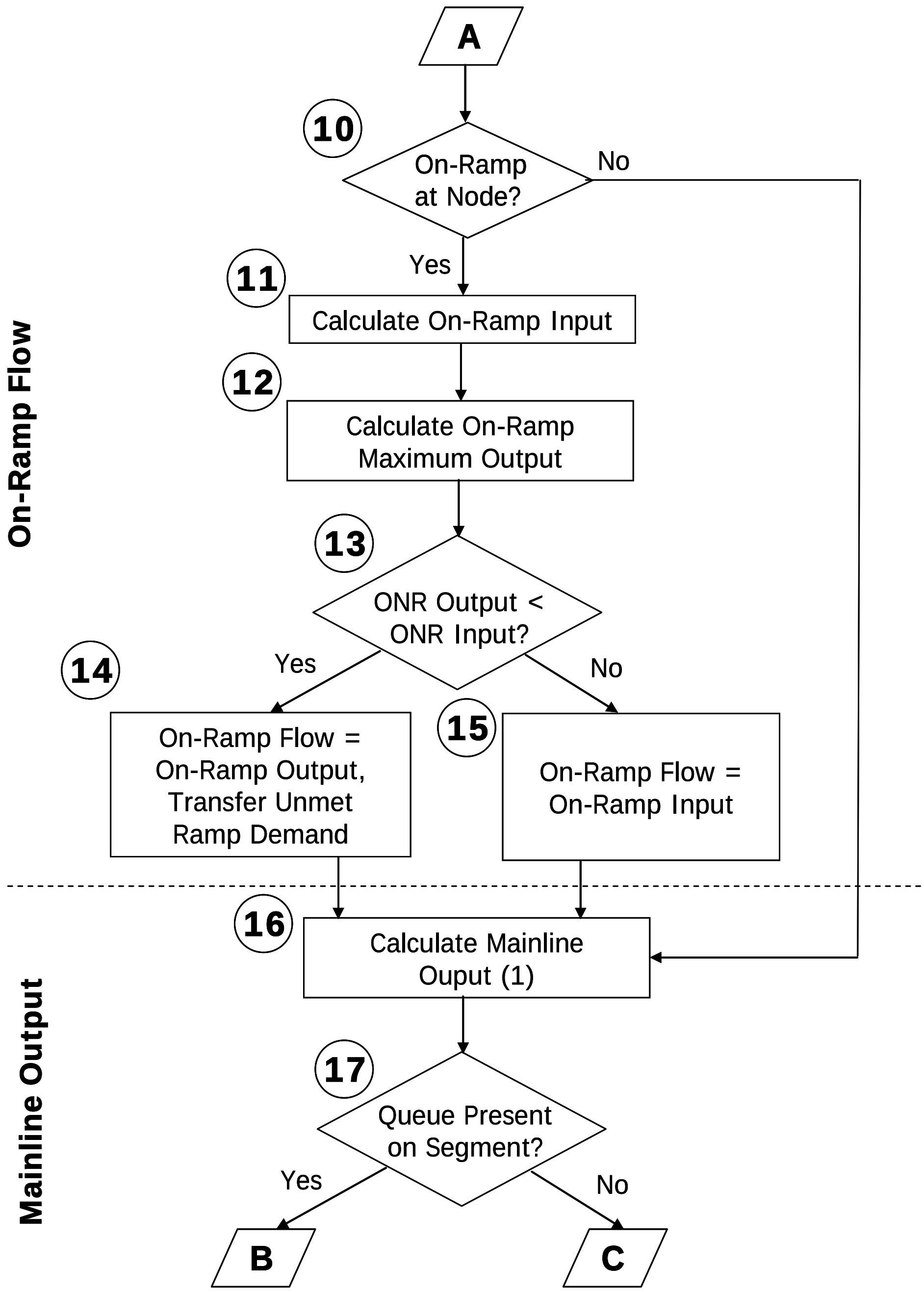


Exhibit 25-6 (cont'd.)
Oversaturated Analysis Procedure

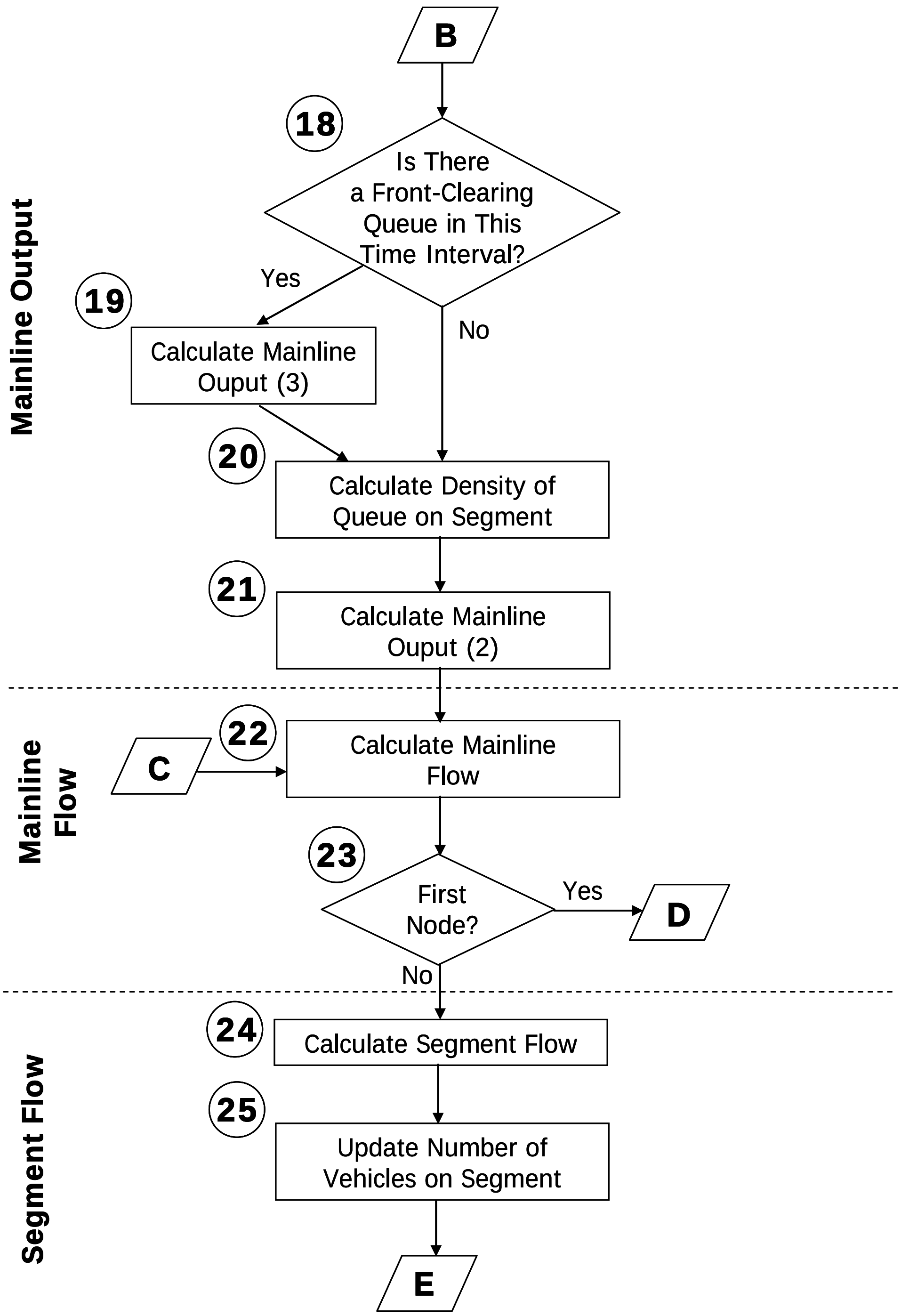
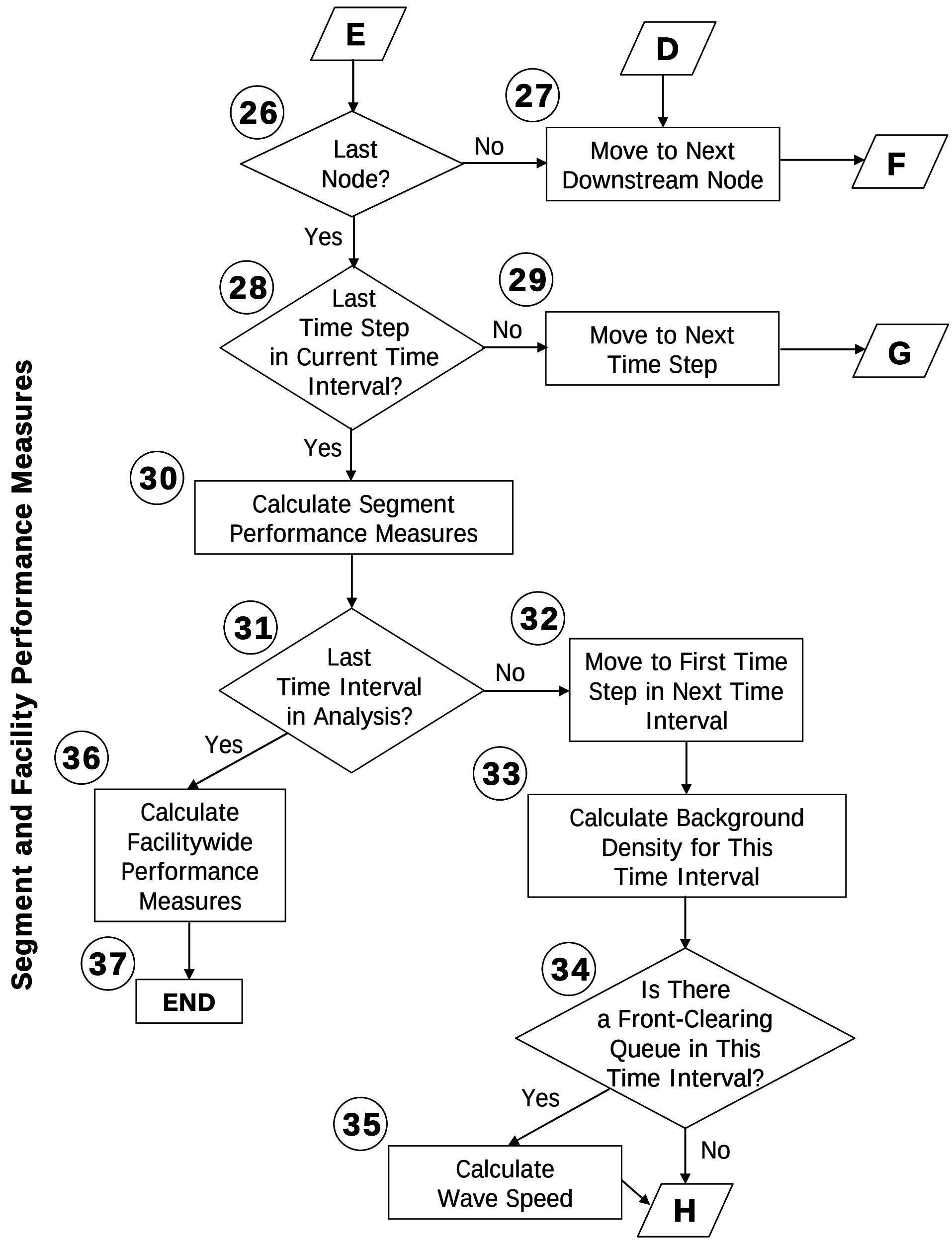


Exhibit 25-6 (cont'd.)
Oversaturated Analysis Procedure



Segment Initialization (Exhibit 25-6, Steps 1–4)

Steps 1 through 4 of the oversaturated procedure prepare the flow calculations for the first time step and specify return points for later time steps. To calculate the number of vehicles on each segment at the various time steps, the segments must contain the proper number of vehicles before the queuing analysis places unserved vehicles on segments. The initialization of each segment is described below. A simplified queuing analysis is initially performed to account for the effects of upstream bottlenecks. These bottlenecks meter traffic downstream of their location. The storage of unserved vehicles (those unable to enter the bottleneck) on upstream segments is performed in a later module. To obtain the proper number of vehicles on each segment, the expected demand *ED* is calculated. *ED* is based on demands for and capacities of the segment and

includes the effects of all upstream segments. The expected demand is the flow of traffic expected to arrive at each segment if all queues were stacked vertically (i.e., no upstream effects of queues). In other words, all segments upstream of a bottleneck have expected demands equal to their actual demand. The expected demand of the bottleneck segment and all further downstream segments is calculated by assuming a capacity constraint at the bottleneck, which meters traffic to downstream segments. The expected demand is calculated for each segment with Equation 25-3:

$$ED(i,p) = \min\left[SC(i,p), ED(i-1,p) + ONRD(i,p) - OFRD(i,p)\right]$$

Equation 25-3

The segment capacity *SC* applies to the length of the segment. With the expected demand calculated, the background density *KB* can be obtained for each segment by using the appropriate segment density estimation procedures in Chapters 11, 12, and 13. The background density is used to calculate the number of vehicles *NV* on each segment by using Equation 25-4. If there are unserved vehicles at the end of the preceding time interval, the unserved vehicles *UV* are transferred to the current time interval. Here, *S* refers to the last time step in the preceding time interval. The (0) term in *NV* represents the start of the first time step in time interval *p*. The corresponding term at the end of the time step is *NV*(*i*, 1, *p*).

$$NV(i,0,p) = KB(i,p) \times L(i) + UV(i,S,p-1)$$

Equation 25-4

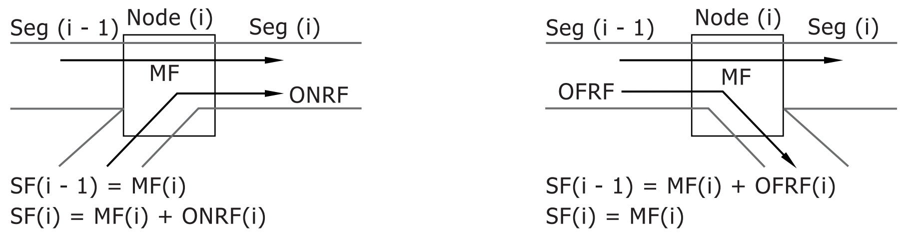
The number of vehicles calculated from the background density is the minimum number of vehicles that can be on the segment at any time. This is a powerful check on the methodology because the existence of queues downstream cannot reduce this minimum. Rather, the segment can only store additional vehicles. The storage of unserved vehicles is determined in the segment flow calculation module later in this chapter.

Mainline Flow Calculations (Exhibit 25-6, Steps 9 and 16–23)

The description of ramp flows follows the description of mainline flows. Thus, Steps 5–8 and 10–15 are skipped at this time to focus first on mainline flow computations. Because of skipping steps in the descriptions, some computations may include variables that have not been described but that have already been calculated in the flowchart.

Flows analyzed in oversaturated conditions are calculated every time step and are expressed in terms of vehicles per time step. The procedure separately analyzes the flow across a node on the basis of the origin and destination of the flow across the node. The mainline flow is defined as the flow passing from upstream segment $i-1$ to downstream segment *i*. It does not include the on-ramp flow. The flow to an off-ramp is the off-ramp flow. The flow from an on-ramp is the on-ramp flow. Each of these flows is shown in Exhibit 25-7 with the origin, destination, and relationship to segment *i* and node *i*.

Exhibit 25-7
Definitions of Mainline and Segment Flows



The segment flow is the total output of a segment, as shown in Exhibit 25-7. Segment flows are calculated by determining the mainline and ramp flows. The mainline flow is calculated as the minimum of six constraints: mainline input (*MI*), *MO1*, *MO2*, *MO3*, upstream segment $i - 1$ capacity, and downstream segment i capacity, as explained next.

Mainline Input (Exhibit 25-6, Step 9)

MI is the number of vehicles that wish to travel through a node during the time step. The calculation includes (*a*) the effects of bottlenecks upstream of the analysis node, (*b*) the metering of traffic during queue accumulation, and (*c*) the presence of additional traffic during upstream queue discharge.

MI is calculated by taking the number of vehicles entering the node upstream of the analysis node, adding on-ramp flows or subtracting off-ramp flows, and adding the number of unserved vehicles on the upstream segment. This is the maximum number of vehicles that wish to enter a node during a time step. *MI* is calculated by using Equation 25-5, where all values have units of vehicles per time step.

Equation 25-5

$$MI(i,t,p) = MF(i-1,t,p) + ONRF(i-1,t,p) - OFRF(i,t,p) + UV(i-1,t-1,p)$$

Mainline Output (Exhibit 25-6, Steps 16–21)

The mainline output is the maximum number of vehicles that can exit a node, constrained by downstream bottlenecks or by merging on-ramp traffic. Different constraints on the output of a node result in three separate types of mainline outputs (*MO1*, *MO2*, and *MO3*).

Mainline Output 1—Ramp Flows (Exhibit 25-6, Step 16)

MO1 is the constraint caused by the flow of vehicles from an on-ramp. The capacity of an on-ramp segment is shared by two competing flows. This on-ramp flow limits the flow from the mainline through this node. The total flow that can pass the node is estimated as the minimum of the segment i capacity and the mainline outputs from the preceding time step. The sharing of Lane 1 (shoulder lane) capacity is determined in the calculation of the on-ramp. *MO1* is calculated by using Equation 25-6.

Equation 25-6

$$MO1(i,t,p) = \min \begin{Bmatrix} SC(i,t,p) - ONRF(i,t,p) \\ MO2(i,t-1,p) \\ MO3(i,t-1,p) \end{Bmatrix}$$

Mainline Output 2—Segment Storage (Exhibit 25-6, Steps 20 and 21)

The second constraint on the output of mainline flow through a node is caused by the growth of queues on a downstream segment. As a queue grows on a segment, it may eventually limit the flow into the current segment once the boundary of the queue reaches the upstream end of the segment. The boundary of the queue is treated as a shock wave. *MO2* is a limit on the flow exiting a node due to the presence of a queue on the downstream segment.

The *MO2* limitation is determined first by calculating the maximum number of vehicles allowed on a segment at a given queue density. The maximum flow that can enter a queued segment is the number of vehicles that leave the segment plus the difference between the maximum number of vehicles allowed on the segment and the number of vehicles already on the segment. The density of the queue is calculated by using Equation 25-7 for the linear density–flow relationship shown in Exhibit 25-5 earlier.

$$KQ(i,t,p) = KJ - [(KJ - KC) \times SF(i,t-1,p)] / SC(i,p)$$

Equation 25-7

Once the queue density is computed, *MO2* can be computed by using Equation 25-8.

$$MO2(i,t,p) = SF(i,t-1,p) - ONRF(i,t,p) + [KQ(i,t,p) \times L(i)] - NV(i,t-1,p)$$

Equation 25-8

Performance of the downstream node is estimated by taking the performance during the preceding time step. This estimation remains valid when there are no interacting queues. When queues do interact and the time steps are small enough, the error in the estimations is corrected in the next time step.

Mainline Output 3—Front-Clearing Queues (Exhibit 25-6, Steps 17–19)

The final constraint on exiting mainline flows at a node is caused by downstream queues clearing from their downstream end. These front-clearing queues are typically caused by incidents in which there is a temporary reduction in capacity. A queue will clear from the front if two conditions are satisfied. First, the segment capacity (minus the on-ramp demand if present) for this time interval must be greater than the segment capacity (minus the ramp demand if present) in the preceding time interval. The second condition is that the segment capacity minus the ramp demand for this time interval be greater than the segment demand for this time interval. A queue will clear from the front if both conditions in the following inequality (Equation 25-9) are met.

$$\begin{aligned}&\text{If } [SC(i,p) - ONRD(i,p)] > [SC(i,p-1) - ONRD(i,p-1)] \\ &\text{and } [SC(i,p) - ONRD(i,p)] > SD(i,p)\end{aligned}$$

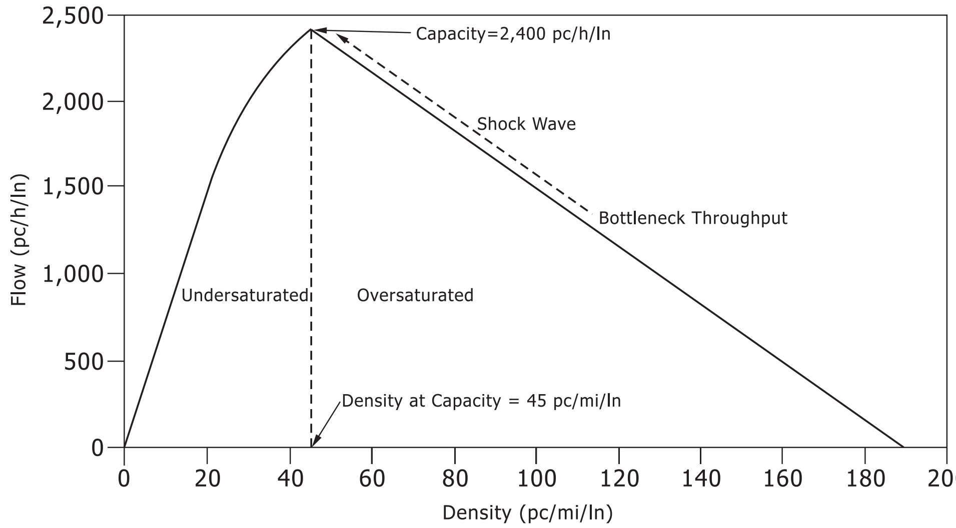
Equation 25-9

A segment with a front-clearing queue will have the number of vehicles stored decrease during recovery, while the back of the queue position is unaffected. Thus, the clearing does not affect the segment throughput until the recovery wave has reached the upstream end of the front-clearing queue. The computational engine implementation is simplified by assuming that the downstream segment is fully queued when the *MO3* constraint is applied. In the flow–density graph shown in Exhibit 25-8, the wave speed is estimated by the

slope of the dashed line connecting the bottleneck throughput and the segment capacity points.

Exhibit 25-8
Flow–Density Function with a Shock Wave

LIVE GRAPH
Click here to view



Note: Assumed *FFS* = 75 mi/h.

The assumption of a linear flow–density function greatly simplifies the calculated wave speed. The bottleneck throughput value is not required to estimate the speed of the shock wave that travels along a known line. All that is required is the slope of the line, which is calculated with Equation 25-10.

Equation 25-10

$$WS(i,p) = SC(i,p) / [N(i,p) \times (KJ - KC)]$$

The wave speed is used to calculate the time it takes the front queue-clearing shock wave to traverse this segment, called the wave travel time, *WTT*. Dividing the wave speed *WS* by the segment length in miles gives *WTT*.

The recovery wave travel time is the time required for the conditions at the downstream end of the current segment to reach the upstream end of the current segment. To place a limit on the current node, the conditions at the downstream node are observed at a time in the past. This time is the wave travel time. This constraint on the current node is *MO3*. The calculation of *MO3* uses Equation 25-11 and Equation 25-12. If the wave travel time is not an integer number of time steps, then the weighted average performance of each variable is taken for the time steps nearest the wave travel time. This method is based on a process described elsewhere (*5–7*).

Equation 25-11

$$WTT = T \times L(i) / WS(i,p)$$

Equation 25-12

$$MO3(i,t,p) = \min \begin{Bmatrix} MO1(i+1, t-WTT, p) \\ MO2(i+1, t-WTT, p) + OFRF(i+1, t-WTT, p) \\ MO3(i+1, t-WTT, p) + OFRF(i+1, t-WTT, p) \\ SC(i, t-WTT, p) \\ SC(i+1, t-WTT, p) + OFRF(i+1, t-WTT, p) \end{Bmatrix} - ONRF(i,t,p)$$

Mainline Flow (Exhibit 25-6, Steps 22 and 23)

The flow across a node is called the mainline flow *MF* and is the minimum of the following variables: *MI*, *MO1*, *MO2*, *MO3*, upstream segment $i - 1$ capacity, and downstream segment i capacity, as shown in Equation 25-13.

$$MF(i,t,p) = \min \begin{Bmatrix} MI(i,t,p) \\ MO1(i,t,p) \\ MO2(i,t,p) \\ MO3(i,t,p) \\ SC(i,t,p) \\ SC(i-1,t,p) \end{Bmatrix}$$

Equation 25-13

In addition to mainline flows, ramp flows must be analyzed. The presence of mainline queues also affects ramp flows.

On-Ramp Calculations (Exhibit 25-6, Steps 10–15)

On-Ramp Input (Exhibit 25-6, Steps 10 and 11)

The maximum on-ramp input *ONRI* is calculated by adding the on-ramp demand and the number of vehicles queued on the ramp. The queued vehicles are treated as unmet ramp demand that was not served in previous time steps. The on-ramp input is calculated with Equation 25-14.

$$ONRI(i,t,p) = ONRD(i,t,p) + ONRQ(i,t-1,p)$$

Equation 25-14

On-Ramp Output (Exhibit 25-6, Step 12)

The maximum on-ramp output *ONRO* is calculated on the basis of the mainline traffic through the node where the on-ramp is located. The on-ramp output is the minimum of two values. The first is segment i capacity minus *MI*, in the absence of downstream queues. Otherwise, the segment capacity is replaced by the throughput of the queue. This estimation implies that vehicles entering an on-ramp segment will fill Lanes 2 to N (where N is the number of lanes on the current segment) to capacity before entering Lane 1. This assumption is consistent with the estimation of V_{12} from Chapter 13, Freeway Merge and Diverge Segments.

The second case occurs when the Lane 1 flow on segment i is greater than one-half of the Lane 1 capacity. At this point, the on-ramp maximum output is set to one-half of Lane 1 capacity. This implies that when the demands from the freeway and the on-ramp are very high, there will be forced merging in a one-to-one fashion on the freeway from the freeway mainline and the on-ramp in Lane 1. An important characteristic of traffic behavior is that, in a forced merging situation, ramp and right-lane freeway vehicles will generally merge one on one, sharing the capacity of the rightmost freeway lane (*8*). In all cases, the on-ramp maximum output is also limited to the physical ramp road capacity and the ramp-metering rate, if present. The maximum on-ramp output is an important limitation on the ramp flow. Queuing occurs when the combined demand from the upstream segment and the on-ramp exceeds the throughput of the ramp

segment. The queue can be located on the upstream segment, on the ramp, or on both and depends on the on-ramp maximum output. Equation 25-15 determines the value of the maximum on-ramp output.

Equation 25-15

$$ONRO(i,t,p) = \min\left\{\begin{array}{l} RM(i,t,p) \\ ONRC(i,t,p) \\ \max\left[\begin{array}{l} \min\left\{\begin{array}{l} SC(i,t,p) \\ MF(i+1,t-1,p) + ONRF(i,t-1,p) \\ MO3(i,t-1,p) + ONRF(i,t-1,p) \end{array}\right\} - MI(i,t,p) \\ \min\left\{\begin{array}{l} SC(i,t,p) \\ MF(i+1,t-1,p) + ONRF(i,t-1,p) \\ MO3(i,t-1,p) + ONRF(i,t-1,p) \end{array}\right\} / 2N(i,p) \end{array}\right] \end{array}\right\}$$

This model incorporates the maximum mainline output constraints from downstream queues, not just the segment capacity. This fact is significant because as a queue spills over an on-ramp segment, the flow through Lane 1 is constrained. This, in turn, limits the flow that can enter Lane 1 from the on-ramp. The values of *MO2* and *MO3* for this time step are not yet known, so they are estimated from the preceding time step. This estimation is one rationale for using small time steps. If there is forced merging during the time step when the queue spills back over the current node, the on-ramp will discharge more than its share of vehicles (i.e., more than 50% of the Lane 1 flow). This situation will cause the mainline flow past node *i* to be underestimated. But during the next time step, the on-ramp flow will be at its correct flow rate, and a one-to-one sharing of Lane 1 will occur.

On-Ramp Flows, Queues, and Delays (Exhibit 25-6, Steps 13–15)

Finally, the on-ramp flow is calculated on the basis of the on-ramp input and output values computed above. If the on-ramp input is less than the on-ramp output, then the on-ramp demand can be fully served in this time step and Equation 25-16 is used.

Equation 25-16

$$ONRF(i,t,p) = ONRI(i,t,p)$$

Otherwise, the ramp flow is constrained by the maximum on-ramp output, and Equation 25-17 is used.

Equation 25-17

$$ONRF(i,t,p) = ONRO(i,t,p)$$

In the latter case, the number of vehicles in the ramp queue is updated by using Equation 25-18.

Equation 25-18

$$ONRQ(i,t,p) = ONRQ(i,t-1,p) + ONRI(i,t,p) - ONRO(i,t,p)$$

The total delay for on-ramp vehicles can be estimated by integrating the value of on-ramp queues over time. The methodology uses the discrete queue lengths estimated at the end of each interval, *ONRQ*(*i*, *S*, *p*), to produce overall ramp delays by time interval.

Off-Ramp Flow Calculation (Exhibit 25-6, Steps 5–8)

The off-ramp flow is determined by calculating a diverge percentage on the basis of the segment and off-ramp demands. The diverge percentage varies only by time interval and remains constant for vehicles that are associated with a particular time interval. If there is an upstream queue, traffic may be metered to this off-ramp, which will cause a decrease in the off-ramp flow. When the vehicles that were metered arrive in the next time interval, they use the diverge percentage associated with the preceding time interval. A deficit in flow, caused by traffic from an upstream queue meter, creates delays for vehicles destined to this off-ramp and other downstream destinations. The upstream segment flow is used because the procedure assumes that a vehicle destined for an off-ramp is able to exit at the off-ramp once it enters the off-ramp segment. This deficit is calculated with Equation 25-19.

$$DEF(i,t,p) = \max\left\{ \begin{matrix} 0 \\ \left\{ \begin{aligned} &\sum_{X=1}^{p-1} SD(i-1,X) - \sum_{X=1}^{p-1}\sum_{t=1}^{T}\left[MF(i-1,t,X) + ONRF(i-1,t,X)\right] \\ &+ \sum_{t=1}^{t-1}\left[MF(i-1,t,p) + ONRF(i-1,t,p)\right] \end{aligned} \right\} \end{matrix} \right\}$$

Equation 25-19

If there is a deficit, then the off-ramp flow is calculated by using the deficit method. The deficit method is used differently in two different situations. If the upstream mainline flow plus the flow from an on-ramp at the upstream node (if present) is less than the deficit for this time step, then the off-ramp flow is equal to the mainline and on-ramp flows times the off-ramp turning percentage in the preceding time interval, as indicated in Equation 25-20.

$$OFRF(i,t,p) = \left[MF(i-1,t,p) + ONRF(i-1,t,p)\right] \times \left[OFRD(i,p-1)/SD(i-1,p-1)\right]$$

Equation 25-20

However, if the deficit is less than the upstream mainline flow plus the on-ramp flow from an on-ramp at the upstream node (if present), then Equation 25-21 is used. This equation separates the flow into the remaining deficit flow and the balance of the arriving flow.

$$\begin{aligned} OFRF(i,t,p) = {} & DEF(i,t,p) \times \left[OFRD(i,p-1)/SD(i-1,p-1)\right] + \\ & \left[MF(i-1,t,p) + ONRF(i-1,t,p) - DEF(i,t,p)\right] \times \\ & \left[OFRD(i,p)/SD(i-1,p)\right] \end{aligned}$$

Equation 25-21

If there is no deficit, then the off-ramp flow is equal to the sum of upstream mainline flow plus the on-ramp flow from an on-ramp at the upstream node (if present) multiplied by the off-ramp turning percentage for this time interval according to Equation 25-22.

$$OFRF(i,t,p) = \left[MF(i-1,t,p) + ONRF(i-1,t,p)\right] \times \left[OFRD(i,p)/SD(i-1,p)\right]$$

Equation 25-22

The procedure does not incorporate any delay or queue length computations for off-ramps.

Segment Flow Calculation (Exhibit 25-6, Steps 24 and 25)

The segment flow is the number of vehicles that flow out of a segment during the current time step. These vehicles enter the current segment either to the mainline or to an off-ramp at the current node. The vehicles that entered the upstream segment may or may not have become queued within the segment. The segment flow is calculated with Equation 25-23.

Equation 25-23

$$SF(i-1,t,p) = MF(i,t,p) + OFRF(i,t,p)$$

The number of vehicles on each segment is calculated on the basis of the number of vehicles that were on the segment in the preceding time step, the number of vehicles that entered the segment in this time step, and the number of vehicles that leave the segment in this time step. Because the number of vehicles that leave a segment must be known, the number of vehicles on the current segment cannot be determined until the upstream segment is analyzed. The number of vehicles on each segment is calculated with Equation 25-24.

Equation 25-24

$$NV(i-1,t,p) = NV(i-1,t-1,p) + MF(i-1,t,p) + ONRF(i-1,t,p) - MF(i,t,p) - OFRF(i,t,p)$$

The number of unserved vehicles stored on a segment is calculated as the difference between the number of vehicles on the segment and the number of vehicles that would be on the segment at the background density. The number of unserved vehicles stored on a segment is calculated with Equation 25-25.

Equation 25-25

$$UV(i-1,t,p) = NV(i-1,t,p) - \left[KB(i-1,p) \times L(i-1)\right]$$

SEGMENT AND RAMP PERFORMANCE MEASURES

In the last time step of a time interval, the segment flows are averaged over the time interval, and the performance measures for each segment are calculated. If there was no queue on a particular segment during the entire time interval, then the performance measures are calculated from the corresponding Chapter 11, 12, or 13 method for that segment. Since there are T time steps in an hour, the average segment flow rate in vehicles per hour in time interval p is calculated by using Equation 25-26.

Equation 25-26

$$SF(i,p) = \frac{T}{S}\sum_{t=1}^{S} SF(i,t,p)$$

If $T = 60$ (1-min time step) and $S = 15$ (interval = 15 min), then $T/S = 4$. If there was a queue on the current segment in any time step during the time interval, then the segment performance measures are calculated in three steps. First, the average number of vehicles over a time interval is calculated for each segment by using Equation 25-27.

Equation 25-27

$$NV(i,p) = \frac{1}{S}\sum_{t=1}^{S} NV(i,t,p)$$

Next, the average segment density is calculated by taking the average number of vehicles NV for all time steps in the time interval and dividing it by the segment length (see Equation 25-28).

$$K(i,p)=\frac{NV(i,p)}{L(i)}$$

Equation 25-28

Next, the average speed on the current segment i during the current time interval p is calculated with Equation 25-29.

$$U(i,p)=\frac{SF(i,p)}{K(i,p)}$$

Equation 25-29

Additional segment performance measures can be derived from the basic measures shown in Equation 25-26 through Equation 25-29. Most prominent is segment delay, which can be computed as the difference in segment travel time at speed $U(i, p)$ and at the segment free-flow speed.

The final segment performance measure is the length of the queue at the end of the time interval (i.e., Step S in time interval p). The length of a queue on the segment, in feet, is calculated with Equation 25-30.

$$Q(i,p)=\frac{UV(i,S,p)}{KQ(i,S,p)-KB(i,p)}\times 5{,}280$$

Equation 25-30

Queue length on on-ramps can also be calculated. A queue will form on the on-ramp roadway only if the flow is limited by a metering rate or by the merge area capacity. If the flow is limited by the ramp capacity, then unserved vehicles will be stored upstream of the ramp roadway, most likely a surface street. The methodology does not account for this delay, since vehicles cannot enter the ramp roadway. However, the unserved vehicles in this case are transferred as added demand in subsequent time intervals. If the queue is on the ramp roadway, the queue length is calculated by using the difference in background density and queue density. For an on-ramp, the background density is assumed to be the density at capacity and the queue density is calculated with Equation 25-31. For on-ramp queue length, Equation 25-31 is used.

$$ONRQL(i,S,p)=\frac{ONRQ(i,S,p)}{KJ-\dfrac{\min\{RM(i,p),ONRO(i,S,p)\times(KJ-KC)\}}{ONRC(i,p)}}$$

Equation 25-31

6. DIRECTIONAL FACILITY MODULE

The previously discussed traffic performance measures can be aggregated over the length of the directional freeway facility, over the time duration of the study interval, or over the entire time–space domain. Each is discussed in the following paragraphs.

Aggregating the estimated traffic performance measures over the entire length of the freeway facility provides facilitywide estimates for each time interval. Facilitywide travel times, vehicle distance of travel, and vehicle hours of travel and delay can be computed, and patterns of their variation over the connected time intervals can be assessed. The FREEVAL-2010 computational engine is limited to 15-min time intervals and 1-min time steps.

Aggregating the estimated traffic performance measures over the time duration of the study interval provides an assessment of the performance of each segment along the freeway facility. Average and cumulative distributions of speed and density for each segment can be determined, and patterns of the variation over connected freeway segments can be compared. Average trip times, vehicle distance of travel, and vehicle hours of travel are easily assessed for each segment and compared.

Aggregating the estimated traffic performance measures over the entire time–space domain provides an overall assessment over the study interval time duration. Overall average speeds, average trip times, total vehicle distance traveled, and total vehicle hours of travel and delay are the most obvious overall traffic performance measures. Equation 25-32 through Equation 25-35 show how the facilitywide performance measures are calculated.

Facility space mean speed in time interval p:

Equation 25-32

$$SMS(NS,p)=\frac{\sum_{i=1}^{NS} SF(i,p)\times L(i)}{\sum_{i=1}^{NS} SF(i,p)\times \frac{L(i)}{U(i,p)}}$$

Average facility density in time interval p:

Equation 25-33

$$K(NS,p)=\frac{\sum_{i=1}^{NS} K(i,p)\times L(i)}{\sum_{i=1}^{NS} L(i)N(i,p)}$$

Overall space mean speed across all intervals:

Equation 25-34

$$SMS(NS,P)=\frac{\sum_{p=1}^{P}\sum_{i=1}^{NS} SF(i,p)L(i)}{\sum_{p=1}^{P}\sum_{i=1}^{NS} SF(i,p)\frac{L(i)}{U(i,p)}}$$

Overall average density across all intervals:

$$K(NS,P)=\frac{\sum_{p=1}^{P}\sum_{i=1}^{NS}K(i,p)\times L(i)}{\sum_{p=1}^{P}\sum_{i=1}^{NS}L(i)N(i,p)}$$

Equation 25-35

These performance measures can be compared for different alternatives to assess the impacts of different volume scenarios or the effects of geometric improvements to the facility.

7. COMPUTATIONAL ENGINE DOCUMENTATION

This section focuses on how the freeway facilities methodology is implemented in the FREEVAL computational engine. It presents the layout of the engine in the form of flowcharts and linkage lists.

FLOWCHARTS

The methodology flowchart is shown above in Exhibit 25-1. A detailed overview of the oversaturated calculation procedure is presented in Exhibit 25-6. In the FREEVAL computational engine implementation, the methodology consists of seven modules:

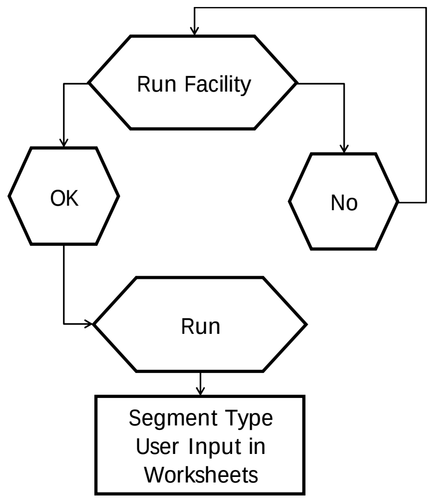
- Opening Dialog Module (Exhibit 25-9)
- Input Formatting Module (Exhibit 25-10)
- Input Revision Module
- Weaving Calculator Input Module (Exhibit 25-11)
- Main Analysis Module (Exhibit 25-12)
- MOE Calculation Module (Exhibit 25-13)
- Supplemental Calculation Module

This subsection provides a separate flowchart for each of these modules, except for the Input Revision and the Supplemental Calculation Modules. The Input Revision Module allows the analyst to revise input data after completing an analysis. Its functionality is thus similar to the Input Formatting Module in terms of reformatting the input–output screens in FREEVAL and then taking the user back to the beginning of the procedure. The Supplemental Calculation Module contains procedures that were removed from the Main Analysis Module because it exceeded coding limits of the programming language. The two modules are therefore discussed concurrently.

The FREEVAL computational engine is composed of a series of worksheets that contain the input and output data for however many time periods the user specifies. Data entry occurs in the actual worksheets as well as in some separate input dialog boxes. The following discussion focuses on the use of computational modules and references the graphical user interface of the engine only when absolutely necessary.

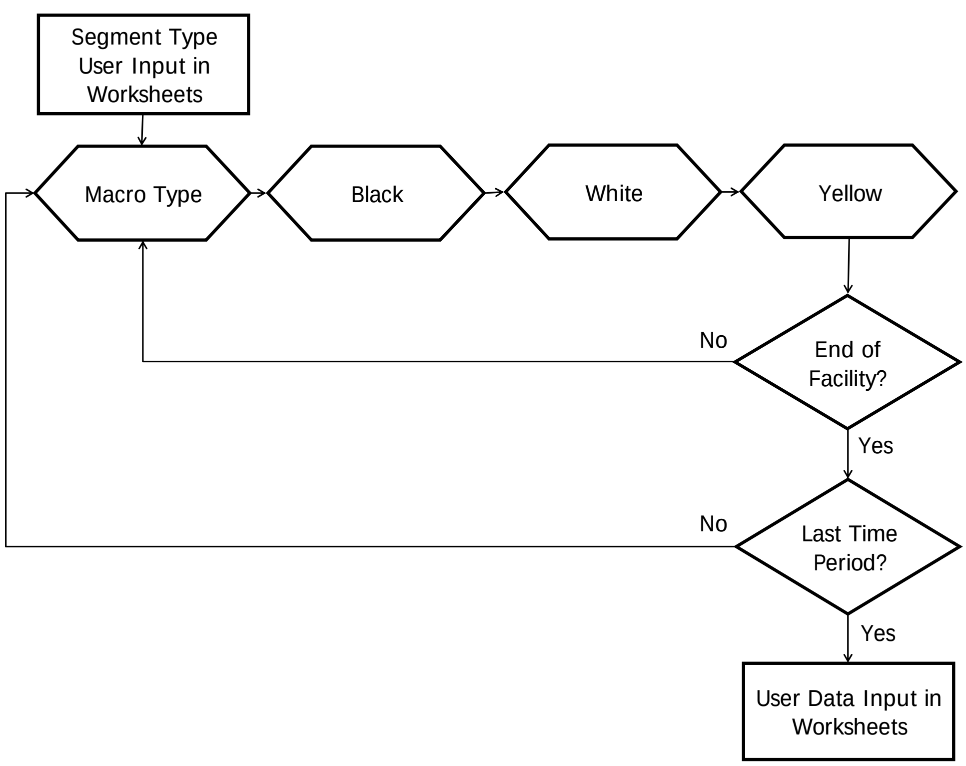
The FREEVAL engine starts with the *Opening Dialog Module* (Exhibit 25-9), which prompts the user to enter some basic information about the facility. The most critical inputs are the number of segments and time periods to be included in the analysis. Upon entering that information and hitting the "Run" button, the user is taken to the first input worksheet, where the facility can be defined in more detail.

Exhibit 25-9
Opening Dialog Module Flowchart



After the user finishes entering all segment types, the "Segment Types Entered" selection button opens the *Input Formatting Module* (Exhibit 25-10). This module serves to make data entry more user-friendly by automatically highlighting required input fields and blacking out unnecessary fields for a specific segment type. The module automatically cycles through all segments and all time periods in the analysis domain.

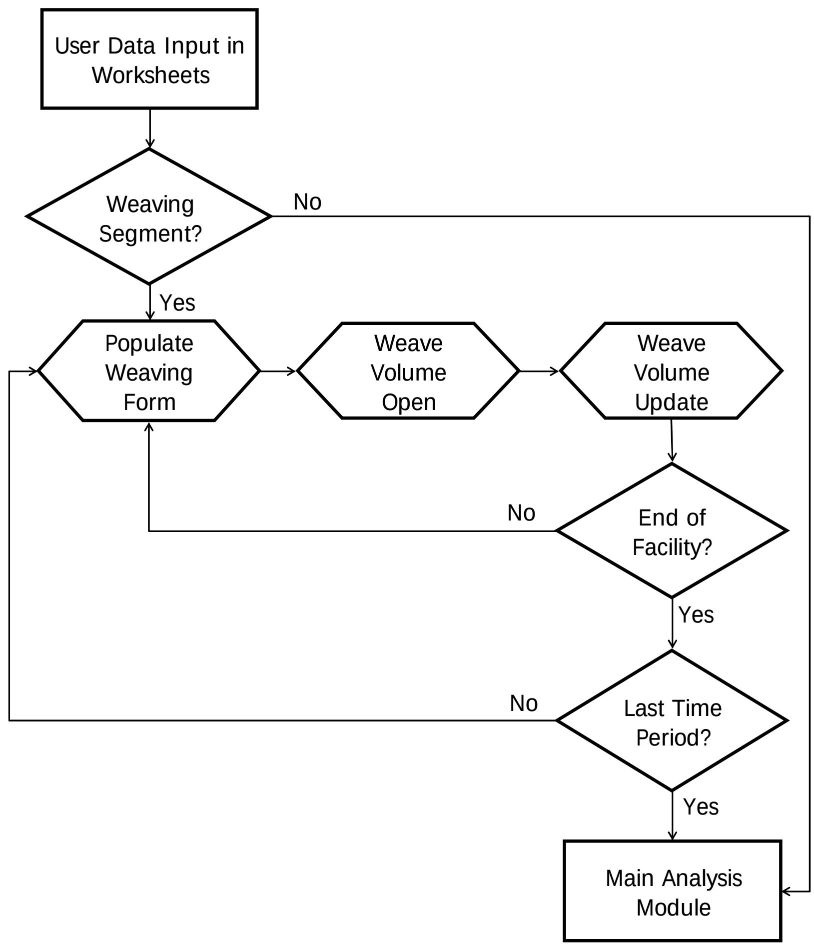
Exhibit 25-10
Input Formatting Module Flowchart



After the input worksheets have been formatted for user input, the analyst enters all necessary input data as discussed in Chapter 10, Freeway Facilities. The input data vary by segment type and are consistent with input requirements in the segment-specific Chapters 11, 12, and 13.

Most input data are entered directly in the input worksheets. The user can further pull up applicable HCM exhibits to obtain heavy-vehicle adjustment factors. They are described more closely in the linkage list section below. Since the input requirements for weaving segments are more complex, an additional input dialog is used for data specific to Chapter 12. The module responsible for the weaving input data is the *Weaving Calculator Input Module* (Exhibit 25-11).

Exhibit 25-11
Weaving Calculator Input Module Flowchart



The module shown in Exhibit 25-11 opens the weaving input for any segments that were coded as a weaving segment and for each time period. The weaving calculator handles all input specific to Chapter 12, including geometry of the weaving segment and volume patterns. The user is required to enter data for each weaving segment and each time period, although geometric data are required only for the first time interval for each weaving segment.

After data entry for all weaving segments has been completed, the *Main Analysis Module* (Exhibit 25-12) is executed. The module follows the Chapter 10 methodology presented in Exhibit 25-1 and contains all computations for undersaturated and oversaturated conditions. The latter contain the oversaturated computation procedure presented in Exhibit 25-6 as well as all queuing and shock-wave calculations discussed in this chapter.

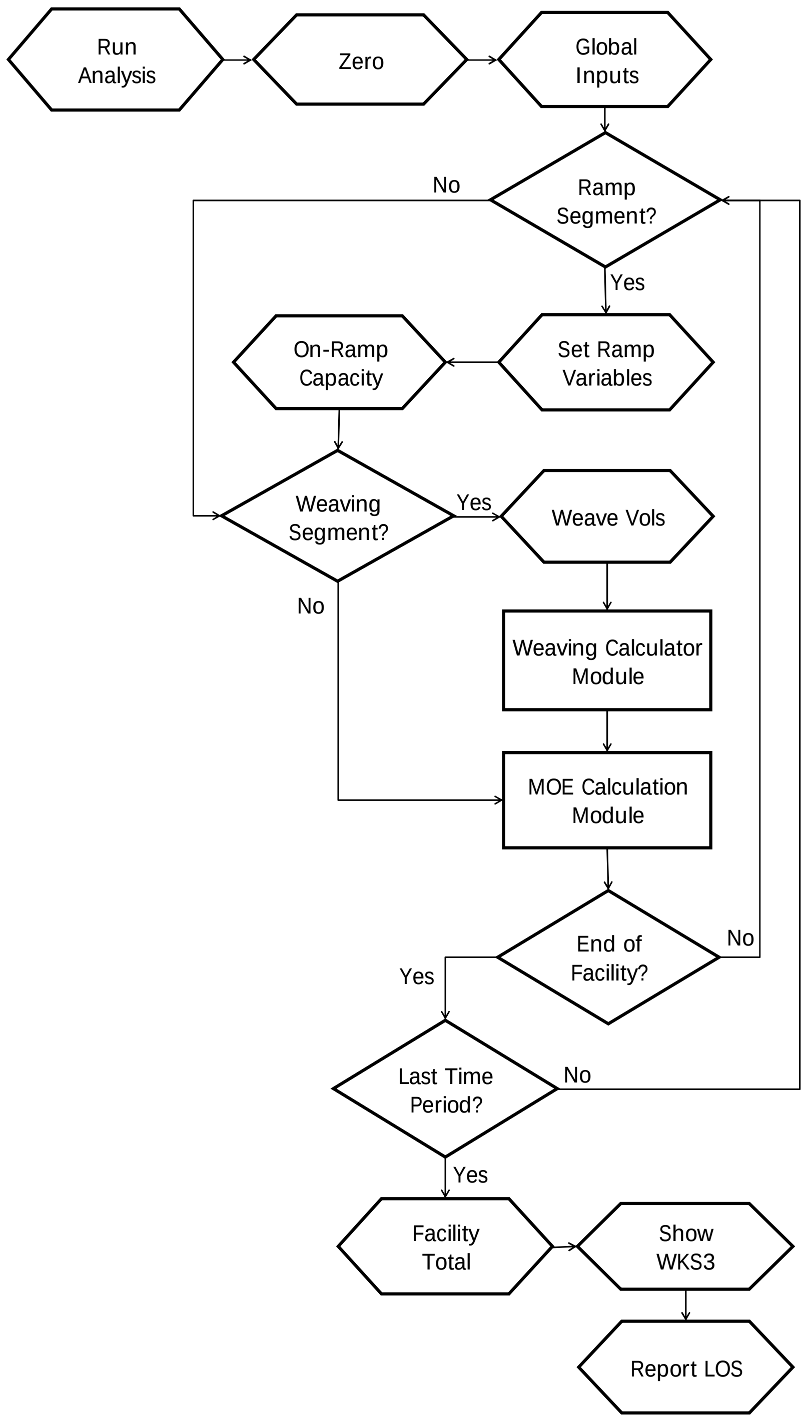
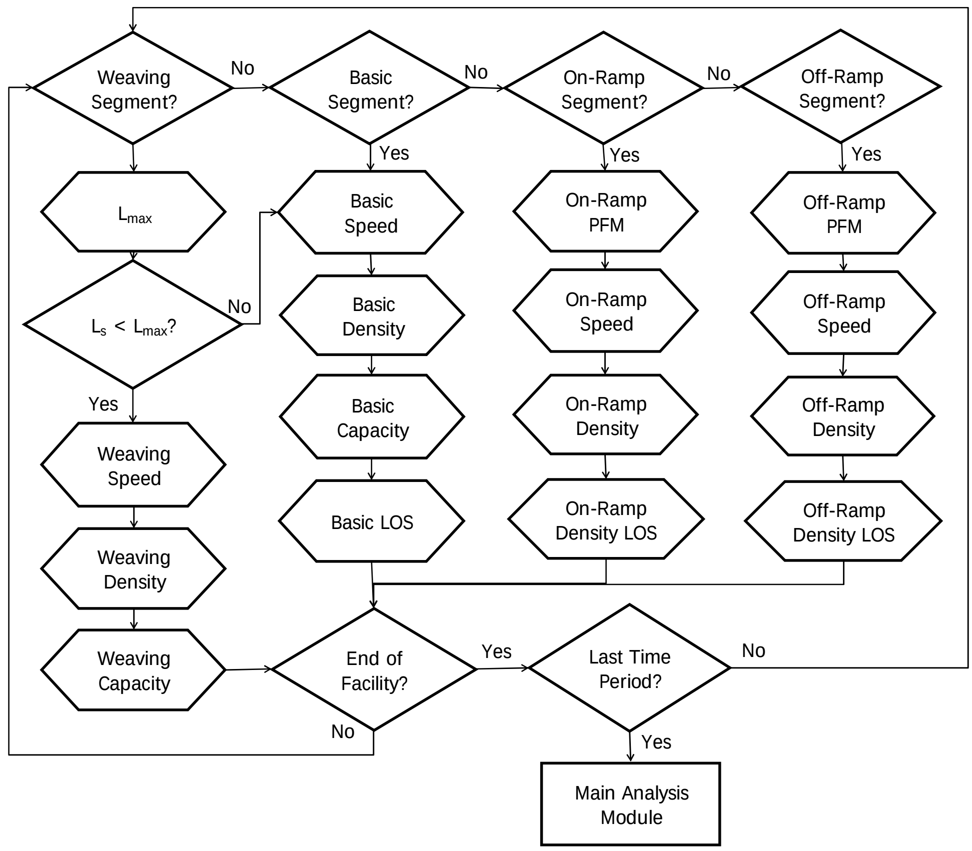


Exhibit 25-12
Main Analysis Module Flowchart

The Main Analysis Module uses the procedure in Exhibit 25-6 for all oversaturated segments and time periods. For undersaturated segments and time periods, the freeway facilities procedure uses the procedure for the different segment types as described in Chapters 11, 12, and 13. The MOEs for these undersaturated procedures are computed by using the *MOE Calculation Module* shown in Exhibit 25-13.

Exhibit 25-13
MOE Calculation Module Flowchart



The individual subroutines contained in the flowcharts shown in Exhibit 25-9 through Exhibit 25-13 are described more closely in the linkage lists below.

The FREEVAL computational engine further contains a series of formatted output worksheets, MOE summary tables, and graphics that are not described in more detail here. For further discussion, the reader is referred to the FREEVAL user guide contained in the Technical Reference Library in Volume 4.

LINKAGE LISTS

This subsection uses linkage lists to describe the main routines that compose the FREEVAL-2010 computational engine. Each table represents one computational module and the subroutines contained within. It further describes conditions for use of each subroutine. The different linkage lists are consistent with the module flowcharts described in the previous section. In particular, the tables describe the following modules:

- Opening Dialog Module (Exhibit 25-14)
- Input Formatting Module (Exhibit 25-15)
- Input Revision Module (Exhibit 25-16)
- Weaving Calculator Input Module (Exhibit 25-17)
- Main Analysis Module (Exhibit 25-18)
- Supplemental Calculation Module (Exhibit 25-19)
- MOE Calculation Module (Exhibit 25-20)

Exhibit 25-14
Opening Dialog Module Routines

Routine	Subroutine	Condition for Use
Run_Facility	Opens an initial dialog box to collect general problem parameters (number of segments, time periods, terrain, etc.).	—
OK	Closes input dialog and opens a verification dialog box before running problem.	—
No	If user clicks "No" in the verification dialog box, the intro page is shown without running the analysis.	—
Run	Collects information from the general dialog box and customizes the spreadsheet by deleting unneeded time periods and segments. Creates output graphs.	Need to have completed initial general dialog box.

Exhibit 25-15
Input Formatting Module Routines

Routine	Subroutine	Condition for Use
Macro Type	Once user inputs segment types, the button that is pressed runs this macro, which customizes the input requirements for each segment, depending on the segment type.	Segment types must be input for each segment (TP1 only).
Black	Makes a cell black if an input variable is not needed for the segment type.	—
White	Makes a cell white if an input variable is needed for the segment type.	—
Yellow	Makes a cell yellow if an input variable is needed for the segment type and default values are not provided.	—

Exhibit 25-16
Input Revision Module Routines

Routine	Subroutine	Condition for Use
Macro 1116	Shows a table with truck equivalencies for various terrain conditions when a button is clicked by the user.	—
Macro 1117	Shows a table with recreational vehicle equivalencies for various terrain conditions when a button is clicked by the user.	—
Input data	Unhides input worksheets once information is entered into the revised input dialog box.	Any revisions to input data (ramp metering, free-flow speed, jam density).
Kj	Opens dialog box when the "Revise Input Data" button is pressed.	—

Exhibit 25-17
Weaving Calculator Input Module Routines

Routine	Subroutine	Condition for Use
Populate Weaving Form	Opens the weaving segment dialog box and enters values from input data for information or defaults.	Input data such as volumes, length, and percent trucks and recreational vehicles. Defaults for weaving parameters.
Weave Vol Open	Makes initial calculations when the volume dialog box is opened.	—
Weave Vol Update	Updates the weaving segment dialog box when a variable is changed.	Calculates other variables based on the variable that has been changed.
Round	Rounds a value to nearest integer.	—

Exhibit 25-18
Main Analysis Module Routines

Routine	Subroutine	Condition for Use
Run Analysis	Runs the analysis, for both under- and oversaturated conditions.	See the methodology flowchart, Exhibit 25-1.
Facility Total	Displays facility total performance measures in the facility total column	Uses internal variables calculated in Run Analysis.
Show WKS3	Shows Worksheet 3, which is the average result for each segment over every time period.	—
Zero	Zeros out some internal variables between time periods.	—
Report LOS	Calculates level of service.	Segment type and density.
Global Inputs	Collects global variables from the introduction dialog box for the methodology.	—
Set Ramp Variables	Used to determine nearest adjacent ramp for ramp analysis.	—
OnRamp Capacity	Calculates capacity of an on-ramp.	On-ramp speed and number of lanes.
Weave Vols	Takes outputs from the weaving form and assigns values to methodology parameters.	—

Exhibit 25-19
Supplemental Calculation Module Routines

Routine	Subroutine	Condition for Use
Number Vehicles	Subroutine pulled out of methodology to compute number of vehicles on a segment.	—
Mainline Flow	Subroutine pulled out of methodology to compute mainline flow for a node.	—

Exhibit 25-20
MOE Calculation Module Routines

Routine	Subroutine	Condition for Use
Basic Speed	Calculates the speed of a basic freeway segment.	See Chapter 11.
Round FFS	Rounds the free-flow speed to nearest 5 mi/h.	—
Basic Density	Calculates the density of a basic segment.	See Chapter 11.
Basic Capacity	Calculates the capacity of a basic segment.	See Chapter 11.
OnRamp Density	Calculates the density of an on-ramp segment.	See Chapter 13.
OnRamp Density LOS	Calculates the density of the ramp influence area for an on-ramp segment.	See Chapter 13.
OnRamp pfm	Calculates P_{fm} for an on-ramp segment.	See Chapter 13.
OnRamp Speed	Calculates the speed of an on-ramp segment.	See Chapter 13.
OFRPfd	Calculates P_{fd} for an off-ramp segment.	See Chapter 13.
OffRamp Density	Calculates the density of an off-ramp segment.	See Chapter 13.
OffRamp Density LOS	Calculates the density of the ramp influence area for an off-ramp segment.	See Chapter 13.
OffRamp Speed	Calculates the speed of an off-ramp segment.	See Chapter 13.
Weaving Speed	Calculates the speed of a weaving segment.	See Chapter 12.
Lmax	Calculates the maximum length of a weaving segment.	See Chapter 12.
Weaving Density	Calculates the density of a weaving segment.	See Chapter 12.
Weaving Capacity	Calculates the capacity of a weaving segment.	See Chapter 12.
LN	Calculates a natural log.	—
Heavy Vehicle	Calculated f_{hv}.	–
Lane Width Reduction	Calculates free-flow speed reduction based on lane width.	See Chapter 11.
LatClear Reduction	Calculates free-flow speed reduction based on lateral clearance.	See Chapter 11.
BasicLOS	Calculates LOS for basic freeway segments.	See Chapter 11.

8. REFERENCES

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2. Eads, B. S., N. M. Rouphail, A. D. May, and F. Hall. Freeway Facilities Methodology in *Highway Capacity Manual* 2000. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1710,* Transportation Research Board, National Research Council, Washington, D.C., 2000, pp. 171–180.
3. Hall, F. L., L. Bloomberg, N. M. Rouphail, B. Eads, and A. D. May. Validation Results for Four Models of Oversaturated Freeway Facilities. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1710,* Transportation Research Board, National Research Council, Washington, D.C., 2000, pp. 161–170.
4. *A Policy on Geometric Design of Highways and Streets,* 5th ed. American Association of State Highway and Transportation Officials, Washington, D.C., 2004.
5. Newell, G. F. A Simplified Theory of Kinematic Waves in Highway Traffic. Part I: General Theory. *Transportation Research,* Vol. 27B, No. 4, 1993, pp. 281–287.
6. Newell, G. F. A Simplified Theory of Kinematic Waves in Highway Traffic. Part II: Queuing at Freeway Bottlenecks. *Transportation Research,* Vol. 27B, No. 4, 1993, pp. 289–303.
7. Newell, G. F. A Simplified Theory of Kinematic Waves in Highway Traffic. Part III: Multidestination Flows. *Transportation Research,* Vol. 27B, No. 4, 1993, pp. 305–313.
8. Newman, L. *Freeway Operations Analysis.* Course Notes. University of California Institute of Transportation Studies University Extension, Berkeley, 1986.

CHAPTER 26
FREEWAY AND HIGHWAY SEGMENTS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. ALTERNATIVE TOOL EXAMPLES FOR BASIC FREEWAY SEGMENTS

General guidance for the use of alternative traffic analysis tools for capacity and level-of-service (LOS) analysis is provided in Chapter 6, HCM and Alternative Analysis Tools.

Chapter 11, Basic Freeway Segments, describes a methodology for analyzing basic freeway segments to estimate their capacity, speed, and density as a function of traffic demand and geometric configuration. **Chapter 26, Freeway and Highway Segments: Supplemental** includes two supplemental example problems that examine situations that are beyond the scope of the Chapter 11 methodology. A typical microsimulation-based tool is used for this purpose and some of the simulation results are compared, where appropriate, with those of the *Highway Capacity Manual* (HCM) procedures.

Both problems are based on Example Problem 3 from Chapter 11, which analyzes a six-lane freeway segment in a growing urban area. The first problem evaluates the facility when a high-occupancy-vehicle (HOV) lane is added, and the second one analyzes operations with an incident within the segment.

The need to determine performance measures based on the analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. The examples presented in Chapter 26 were applied by using versions of simulation packages available at the time of chapter development; these packages were not entirely consistent with the recommended trajectory-based measures described in Chapter 7.

EXAMPLE PROBLEM 1: DEVELOPMENT OF HOV LANE

The inputs for this problem are identical to those of Example 3 in Chapter 11. In the situation studied with that example problem, the existing six-lane freeway was projected to remain at Level of Service (LOS) D in the next 3 years, and demand was projected to exceed capacity within 7 years. An additional question, which would illustrate the use of alternative tools, is, “How would the facility operate if a fourth lane were added for use by HOVs?”

The HCM procedure does not take into consideration HOVs and therefore does not estimate performance measures related to HOV lanes. A simulation tool was used to evaluate this scenario, estimating the capacity and density of the basic freeway segment when an HOV lane is present. Exhibit 26-1 provides a graphical view of the simulated site with the leftmost lane designated as an HOV lane. It is assumed that the HOV lane is separated from the general traffic and that vehicles are not allowed to make lane changes except at designated access points.

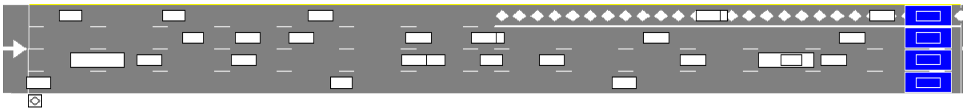

Exhibit 26-1
Graphical View of Basic Freeway Segment with HOV Lane

The subject segment and upstream segment lengths are both 1 mi. On the basis of the inputs given in Example Problem 3 from Chapter 11, it was assumed that the HOV lane would be operational in 4 years. The 3-year peak-hour

demand forecast was 5,600 veh/h; thus, the hourly demand volume in the fourth year was assumed to be 4% higher (using the same growth rate as that in the example problem), or 5,824 veh/h. The peak hourly flow rate was assumed to be 5,824/0.95 = 6,131 veh/h, by using the example problem's peak hour factor of 0.95. The simulation analysis was conducted in units of vehicles per hour; simulation packages typically provide flows in vehicles per hour, and thus there was no conversion to passenger-car equivalents for this analysis.

Next, experiments were conducted to obtain the capacity and density of the subject segment as a function of (*a*) the percent of carpools in the traffic stream, (*b*) the percent of HOVs that use the HOV lane, (*c*) the distance upstream of the beginning of the HOV lane when drivers begin to react and position themselves to use (or avoid) the lane, and (*d*) the percent of HOV violators (i.e., non-HOVs that use the HOV lane). The results of these experiments are provided below. Ten runs were conducted for each experiment.

Exhibit 26-2 provides the overall density of the study segment as well as the densities of the HOV lane and the three general-purpose lanes as a function of the carpool percentage in the traffic stream. This density is estimated for a total demand flow rate of 6,131 veh/h. As expected, the average density on the link remains constant as the carpool percentage increases. When carpools increase, the density of the HOV lane increases, whereas the density of the remaining lanes decreases.

Exhibit 26-2
Density of Basic Freeway Segment with HOV Lane as a Function of Carpool Percentage

LIVE GRAPH
Click here to view

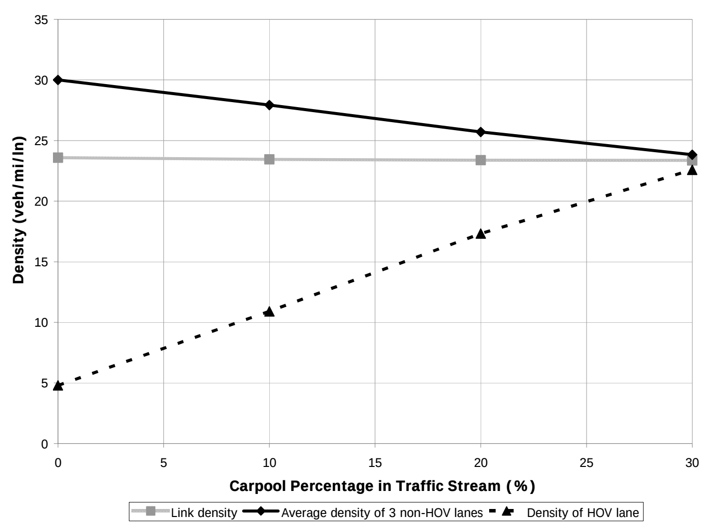

Exhibit 26-3 shows the capacity of the study segment as a function of the carpool percentage in the traffic stream. The capacity was measured as the maximum throughput of the segment when the demand was very high. As shown, the capacity increases more or less linearly as the percent of carpools increases. This increase occurs because when the carpool percentage is low, the HOV lane is underutilized. As the percent of carpools increases, more vehicles use the HOV lane, and the capacity of the entire segment increases as more vehicles shift to the HOV lane.

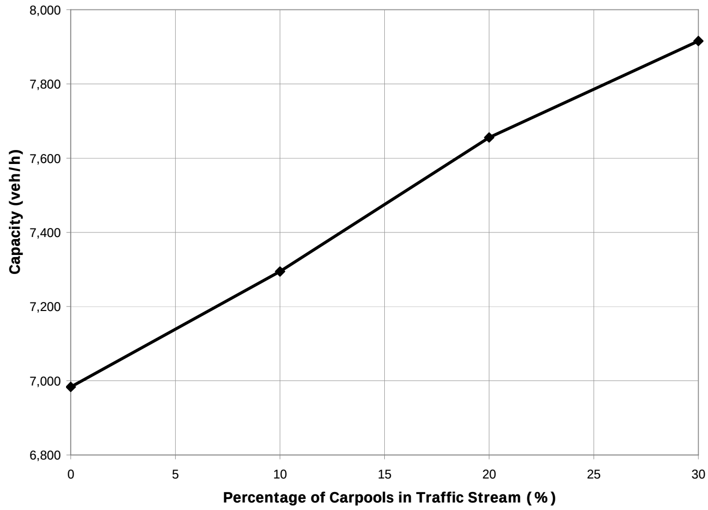

Exhibit 26-3
Capacity of Basic Freeway Segment with HOV Lane as a Function of Carpool Percentage

LIVE GRAPH
Click here to view

Exhibit 26-4 shows the density of the study segment as well as the density of the HOV lane and the other lanes separately as a function of the percent of HOVs using the HOV lane. The simulation package employs the percent of HOVs using the HOV lane as an input, and that number stays constant throughout the simulation, regardless of the operations of the general-use lanes. The average density of the HOV lane is relatively low even when the proportion of HOVs using the HOV lane is 100%. In this experiment, the proportion of carpools is 10%, and therefore the density of the HOV lane remains quite low even if all HOVs in the traffic stream use it.

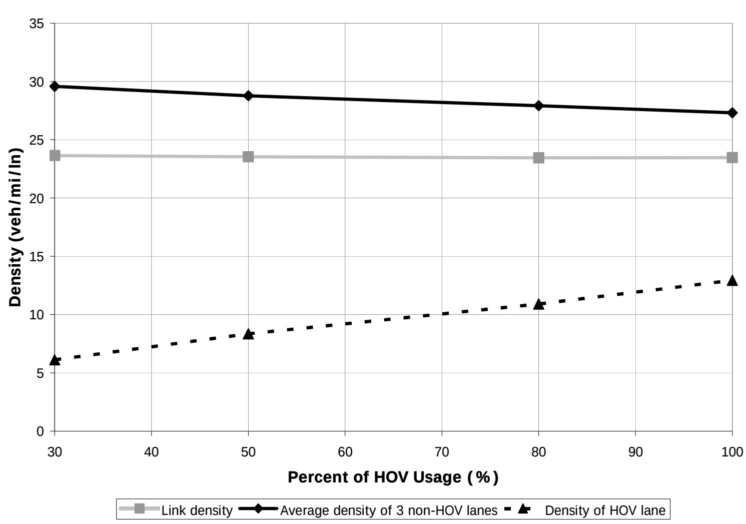

Exhibit 26-4
Density of Basic Freeway Segment with HOV Lane as a Function of Percent of HOV Usage

LIVE GRAPH
Click here to view

Exhibit 26-5 shows the capacity of the study segment as a function of the percent of HOVs using the HOV lane. The capacity was measured as the maximum throughput of the segment when the demand was very high.

Exhibit 26-5
Capacity of Basic Freeway Segment with HOV Lane as a Function of Percent of HOV Usage

LIVE GRAPH
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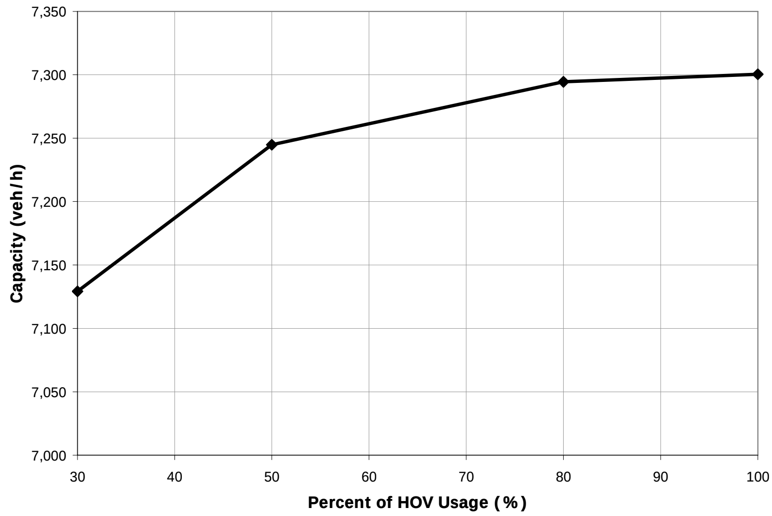

Exhibit 26-6 shows the density of the segment with the HOV lane as a function of the distance when drivers begin to react to the presence of the HOV lane. As before, the simulation package employs the percent of HOVs using the HOV lane as an input, and that number stays constant throughout the simulation regardless of whether there is congestion in the general-use lanes. In this experiment, the proportion of carpools was 10% and the proportion of HOVs using the HOV lane was 80%. As shown, when drivers are given no warning of the HOV lane, the density across all lanes is very similar. This result was surprising and somewhat unrealistic, since it indicated that once in the HOV link, the simulated vehicles do not seem to reposition themselves to use the HOV lane. However, this scenario of not providing adequate warning of the HOV lane's presence is not very realistic in the field.

Exhibit 26-6
Density of Basic Freeway Segment with HOV Lane as a Function of HOV Warning Sign Distance

LIVE GRAPH
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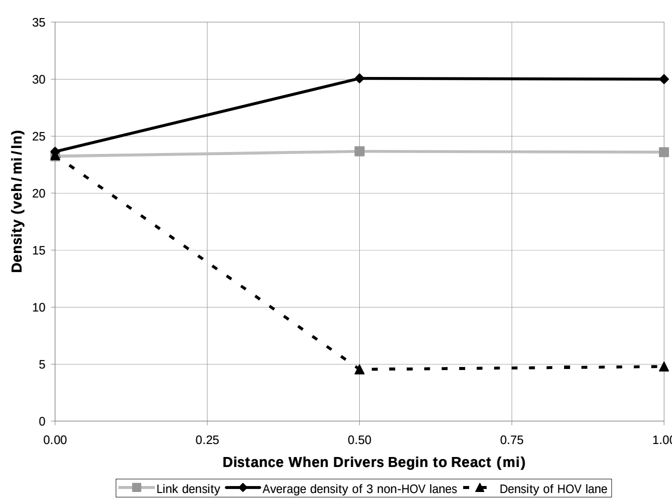

As the distance when drivers begin to react increases, the density at each group of lanes changes significantly. When vehicles are given a warning at least ½ mi upstream of the HOV lane, the density among the lanes is changed

accordingly and does not change when the warning distance increases to farther than ½ mi.

Exhibit 26-7 shows the capacity of the study segment as a function of the distance when drivers begin to react to the presence of the HOV lane. The proportion of carpools for this experiment is 10%. As shown, the capacity decreases when this distance increases. The drop occurs because for the longer distances vehicles have had the opportunity to redistribute themselves among the four lanes, and the HOV lane is used only by carpools. Because the percent of carpools is relatively low, the HOV lane is not as effectively used as when all vehicles were using it, and the overall throughput drops.

Exhibit 26-7
Capacity of Basic Freeway Segment with HOV Lane as a Function of HOV Warning Sign Distance

LIVE GRAPH
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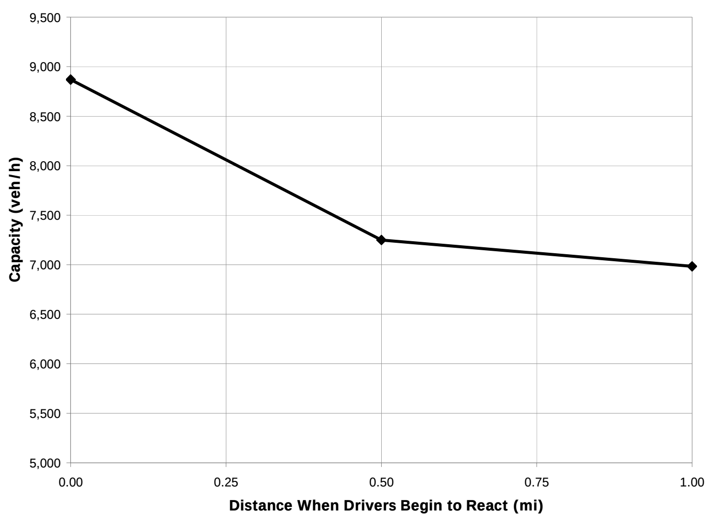


Exhibit 26-8 shows the density as a function of HOV violators (i.e., non-HOVs using the HOV lane). The proportion of carpools is 10%, the distance to warning is 1 mi, and the proportion of HOV usage is 80%. As expected, the density of the HOV lane increases as the percent of violators increases.

Exhibit 26-8
Density of Basic Freeway Segment with HOV Lane as a Function of Percent of HOV Violators

LIVE GRAPH
Click here to view

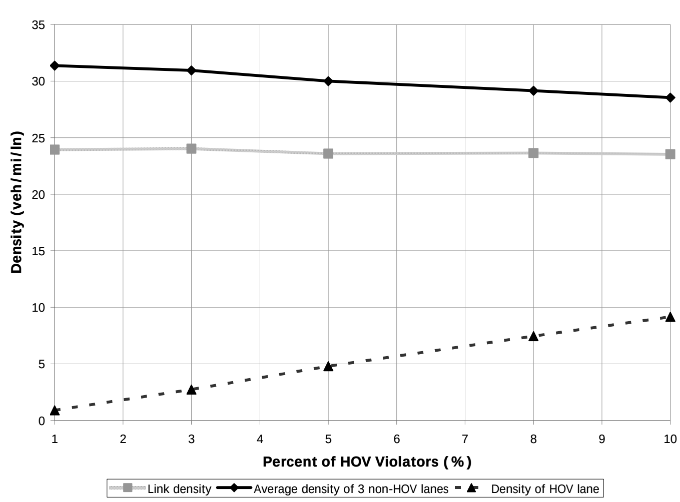
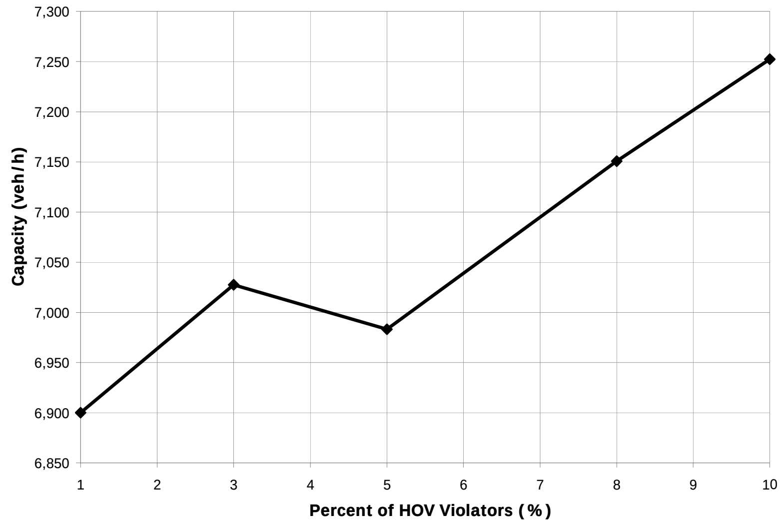


Exhibit 26-9 shows the capacity of the study segment as a function of the percent of HOV violators. The capacity generally increases as the percent of

violators increases, because the HOV lane is generally underutilized, and thus when non-HOVs use it, its throughput increases.

Exhibit 26-9
Capacity of Basic Freeway Segment with HOV Lane as a Function of Percent of HOV Violators



This type of analysis cannot be conducted by using the HCM, since the HCM method does not estimate the HOV-lane density separately. The experiments shown in this example evaluating the potential installation of an HOV lane cannot be conducted with the HCM methods. Variables such as the impact of the distance of the HOV warning sign cannot be evaluated, because they pertain to driver behavior attributes and their impact on density and capacity. The impact of the percent of carpools and the percent of violators could perhaps be estimated with appropriate modifications to the HCM method.

EXAMPLE PROBLEM 2: INCIDENT OCCURRENCE

Example Problem 2 is also based on Example Problem 3 of Chapter 11. It evaluates the operating conditions that would result if a lane-blocking incident occurred along the segment. The HCM procedure for basic freeway segments cannot estimate the impacts of the presence of an incident. A typical simulation tool was used to evaluate this scenario and to estimate the capacity and density of the basic freeway segment when an incident occurs.

The subject segment length is 1 mi, and the upstream segment is also 1 mi. Each simulation run was for 1 full hour, and it was assumed that the incident started 15 min after the beginning of the simulation and—where not varied as part of the experiment—the duration of the incident was 15 min. It was assumed that the demand flow rate was 6,131 veh/h and that one lane would be blocked during the incident. The incident was assumed to occur in the middle of the link, with the rightmost lane blocked.

A series of experiments obtained the subject segment's capacity and density as a function of (*a*) the incident duration and (*b*) the length of the segment affected. Exhibit 26-10 presents the density as a function of the incident duration, and Exhibit 26-11 presents the hourly capacity. As shown, as the incident duration increases, the average density increases, whereas the capacity decreases.

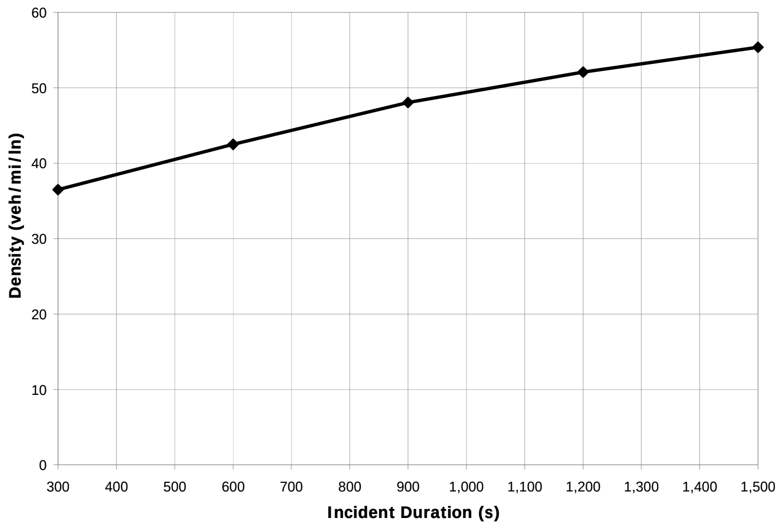


Exhibit 26-10
Density of Basic Freeway Segment for Varying Incident Duration

LIVE GRAPH
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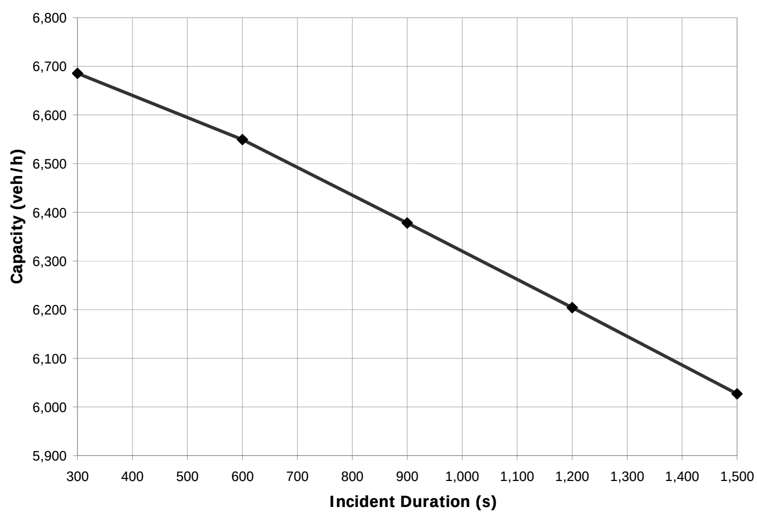

Exhibit 26-11
Capacity of Basic Freeway Segment for Varying Incident Duration

LIVE GRAPH
Click here to view

Exhibit 26-12 presents the density of a basic freeway segment as a function of the length of the segment affected during the incident. The graph provides density based on link statistics (i.e., the density of the link with the incident). As expected, the greater the length of the segment affected, the higher the link density. The change, however, is relatively small—on the order of less than 0.5 veh/mi/ln. The results would be different if density were based on detector output. For example, if detectors were placed downstream of the incident, the density recorded there would not be affected by the length of the segment affected, and the relationship would be flat.

Exhibit 26-12
Link Density of Basic Freeway Segment for Varying Length of Segment Affected

LIVE GRAPH
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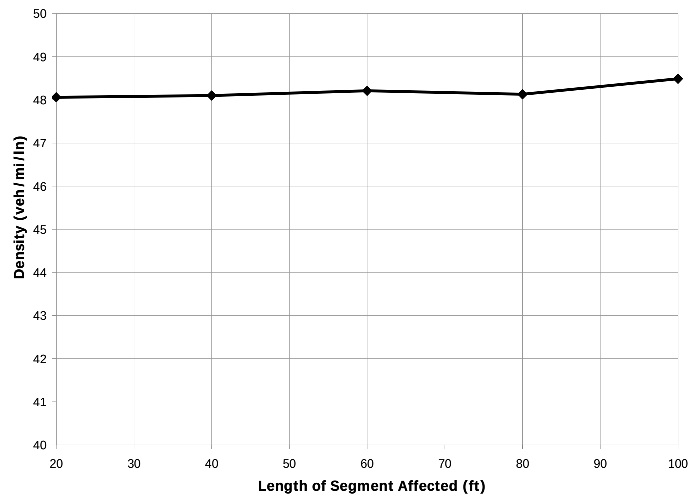

Exhibit 26-13 plots the capacity as a function of the length of the segment affected during the incident. In this graph, capacity is measured at detectors downstream of the incident. As shown, there is a slight increase in capacity as the length of the segment affected increases. This finding may be because the distance from the end of the incident area to the detectors decreases as the length of the segment affected by the incident increases.

Exhibit 26-13
Capacity of Basic Freeway Segment for Varying Length of Segment Affected

LIVE GRAPH
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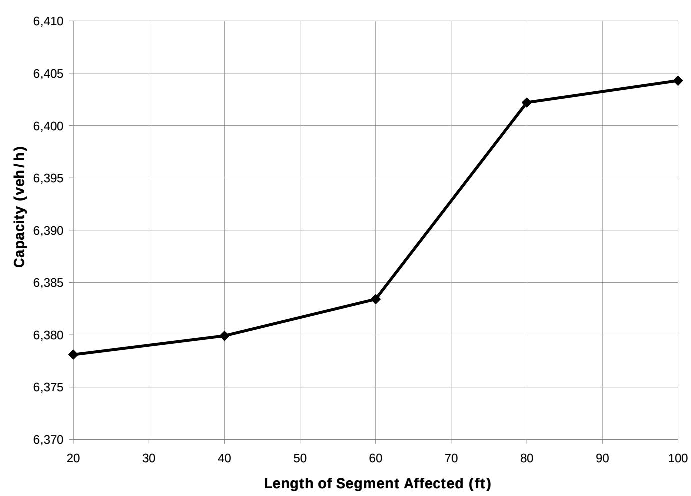


Exhibit 26-14 plots the density of the basic freeway segment as a function of the location of the warning sign upstream of the incident. As shown, the density fluctuates within 1.5 veh/mi/ln, and the general trend for the entire range tested is for the density to decrease as the distance between the warning sign and the incident increases. However, the magnitude of the change is very small. Generally, the presence of the warning sign results in a speed drop as vehicles attempt to change lanes in anticipation of the lane blockage ahead.

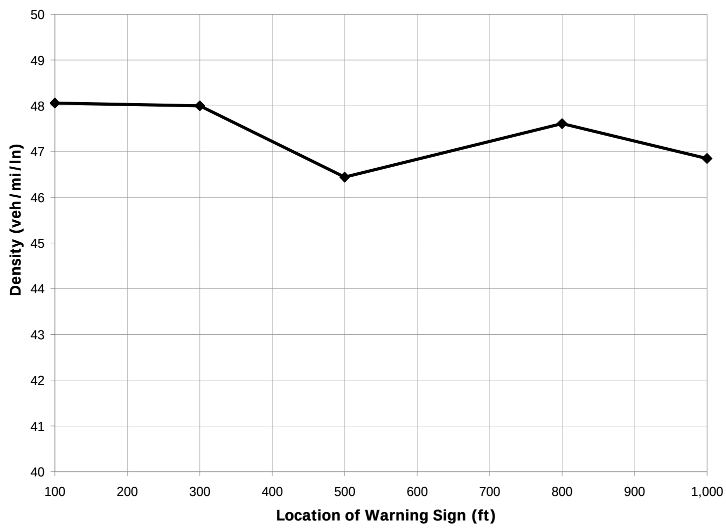


Exhibit 26-14
Density of Basic Freeway Segment for Varying Location of Warning Sign

LIVE GRAPH
Click here to view

Exhibit 26-15 plots the capacity of a basic freeway segment as a function of the location of the warning sign. The results are consistent with those in Exhibit 26-14, indicating that capacity generally is reduced as the distance between the warning sign and the incident increases. Again, this result is potentially related to a speed drop in the traffic stream associated with the presence of a warning sign.

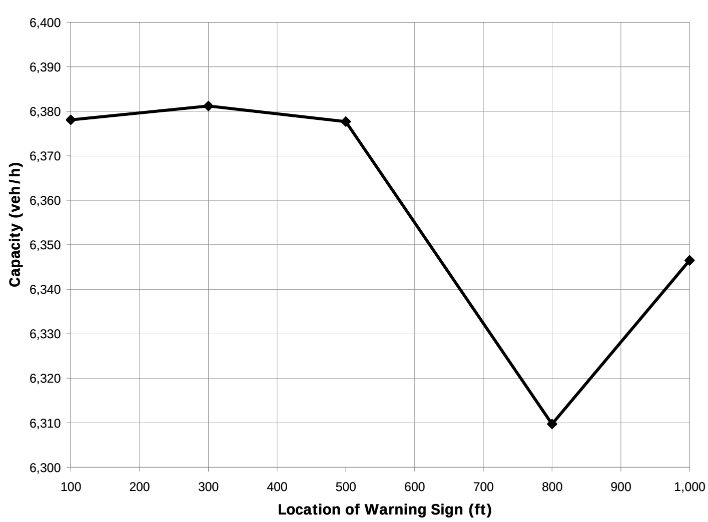


Exhibit 26-15
Capacity of Basic Freeway Segment for Varying Location of Warning Sign

LIVE GRAPH
Click here to view

As indicated earlier, the presence of incidents cannot be evaluated by using the HCM. The freeway facilities methodology in Chapter 10 can handle changes in the segment capacity; however, variables related to driver behavioral aspects cannot be analyzed. In addition, the segment capacity is an input to the freeway facilities methodology, and thus a tool is needed to estimate the segment capacity when an incident occurs.

2. STATE-SPECIFIC HEAVY-VEHICLE DEFAULT VALUES

Research into the percentage of heavy vehicles on uninterrupted-flow facilities (*1*) found a wide range of average values from state to state, so much so that not even regional default values could be developed. Exhibit 26-16 presents default values for the percentage of heavy vehicles on freeways by state and area population, based on data from the 2004 Highway Performance Monitoring System (HPMS). Exhibit 26-17 presents similar default values for multilane and two-lane highways. Where states or local jurisdictions have developed their own values, these may be substituted. Analysts may also wish to develop their own default values based on more recent data.

Exhibit 26-16
State-Specific Default Values for Percentage of Heavy Vehicles on Freeways

State	Rural	Small Urban	Medium Urban	Large Urban	State	Rural	Small Urban	Medium Urban	Large Urban
AL[*]	14[a]	7	7	7[a]	MT	22[c]	16[c]	12[c]	NA
AK	4	5[b]	5	3[b]	NC[*]	19[b]	12[b]	12	10[a]
AR	30	24	13	14	ND	21[c]	22[c]	10[c]	NA
AZ	21	19	18	11	NE	36	37	11	8
CA	16	10	7	6	NH	15[b]	12[b]	6[b]	7[b]
CO	12	10	8	7	NJ	8	6	6	9
CT	13	6	6	5	NM	26	12	21	12
DC	NA	NA	NA	4[b]	NV	34[b]	26	18[b]	11[b]
DE	—	—	9[b]	8[b]	NY	18	11	11	7
FL[*]	11	7	12	6	OH	24	13	10	8
GA[*]	19[b]	7[b]	12	8[b]	OK	28	27	12	10
HI	5	19[b]	2	3	OR	26	19	10	7
IA	20[c]	24[c]	11[c]	10[c]	PA	16	13	9	8
ID	29[c]	28[b]	12[b]	7[b]	PR[*]	6	7[b]	7	4[b]
IL	21	23	16	9	RI	3	—	NA	4
IN	26	25	23	14	SC[*]	19[b]	7[b]	7	8[b]
KS	21[c]	17[c]	8[c]	9[b]	SD	20[c]	14[c]	9[c]	NA
KY[*]	20[a]	16	12	10[a]	TN[*]	19	12	12	8
LA[*]	12[c]	7[b]	12	10[c]	TX	16	28[c]	8	5
MA	7[a]	5	4[a]	4	UT	34[c]	—	18	13
MD	18	14	17	8	VA[*]	9	7	7	4
ME	5	5	5	NA	VT	15	12	6	NA
MI	18	12	13	8	WA	11	10	7	6
MN	11	10	6	4	WI	6	6	6	6
MO	29[b]	23[b]	13[b]	10[b]	WV	16[b]	13[b]	9[b]	NA
MS[*]	9[b]	7[b]	7	6[b]	WY	33[c]	36[a]	28[c,d]	NA

Notes: Rural: <5,000 pop.; small urban: 5,000–50,000; medium urban: 50,000–250,000; large urban: >250,000.

NA = population group does not exist within the state; —, data not available.

Values shown represent mean values for the state for each population type except where noted otherwise.

* Because of limited data, small urban values were combined for these groups of states: AL, MS, PR, SC, and VA and FL, GA, KY, LA, NC, and TN. Medium urban values were combined for AL, FL, and VA.

[a] Reported values appeared to be a mix of field observations and statewide values. The latter were discounted, such that the averages shown here are based primarily on values deemed to be field observations, with some consideration given to nearby states and the value state personnel thought was statewide.

[b] The default value was estimated from field observations from nearby states, because of insufficient field data, a lack of data for this road type, or too-heavy reliance on statewide values.

[c] The peak-period percentage is identical to the daily average percentage for nearly all observations in the HPMS data set. Default values were estimated primarily from the daily average value but took into account the results from nearby states, particularly the difference between peak and daily values in those states.

[d] This distribution was bimodal, with one group centered on 19% and the other on 44%.

Source: *NCHRP Report 599* (*1*).

Exhibit 26-17
State-Specific Default Values for Percentage of Heavy Vehicles on Multilane and Two-Lane Highways

	Two-Lane Highways		Multilane Highways			Two-Lane Highways		Multilane Highways	
State	**Rural**	**Small Urban**	**Rural**	**Small Urban**	**State**	**Rural**	**Small Urban**	**Rural**	**Small Urban**
AL	6[a]	6[a]	4[a]	6[a]	MT	10[*]	4[*]	6[*]	3[*]
AK	10	2	6	3	NC	8[b]	4[b]	6[b]	6[b]
AR	14	7	11	12	ND	14[*]	3[*]	12[*]	7[*]
AZ	9	11	9	9	NE	10	3	12	5
CA	9	5	9	6	NH	6[b]	6[a]	6[b]	6[b]
CO	11	4	5	5	NJ	8	7	8	6[b]
CT	3	3	2	6[b]	NM	17	7	23	12
DC	NA	NA	NA	NA	NV	17[b]	5[*]	10[*]	6[*]
DE	7	6	9	8	NY	8	5	8	5
FL	8	4	7	7	OH	11	4	14	9
GA	8[b]	5[b]	6[b]	6[b]	OK	14[a]	5	17	11
HI	3	3	2	2	OR	12	5	6	9
IA	4[*]	5[*]	5[*]	4[*]	PA	6	3	5	4
ID	12[*]	7[*]	16[*]	9[*]	PR	5	5[b]	5	6
IL	8	5	8	6	RI	2	1	2	6[b]
IN	10	6[a]	12	10	SC	8[b]	5[b]	6[b]	6[b]
KS	15[a]	3	12[*]	6[*]	SD	13[*]	4[*]	12[*]	7[*]
KY	16[a]	6[a]	9[a]	6[a]	TN	5	4[a]	6	4
LA	16[*]	10[*]	6[b]	16	TX	13	9	12	9
MA	3[a]	3[a]	7[b]	6[b]	UT	20[*]	9[*]	22[*]	14[*]
MD	10	6	12	8	VA	4	2	5	2
ME	5	3	4	3	VT	8	5[a]	7	6[b]
MI	9	7[a]	8	4	WA	15	8[a]	10	7
MN	9	8[a]	8	6	WI	4	5[a]	4	5[a]
MO	9[*]	6[*]	12[b]	10[*]	WV	6[b]	6[b]	5[b]	6[b]
MS	14[a]	5[a]	6[b]	6[a]	WY	15[*]	6[*]	10[*]	9[*]

Notes: Rural: <5,000 pop.; small urban: 5,000–50,000.

NA: Population group does not exist within the state

Values shown represent mean values for the state for each population type except where noted otherwise.

* The peak-period percentage is identical to the daily average percentage for all or almost all observations in the HPMS data set for this cell. Default values were estimated primarily from the daily average value for this cell, taking into account the results for other similar states in the same region, and in particular the difference between peak and daily average values in those states.

[a] Reported values appeared to be a mix of field observations and statewide values. The latter were discounted, such that the averages shown here are based primarily on values deemed to be field observations, with some consideration given to nearby states and the value the state personnel thought was statewide.

[b] Either there are insufficient field data, such that regional averages were used, or there are no usable field data, either because there are no data in the state for this road type or because there is too heavy a reliance on statewide values for both the peak period and the daily average. In these cases, the default value was estimated from field observations for nearby states.

Source: *NCHRP Report 599* (*1*).

3. REFERENCE

This report can be found in the Technical Reference Library in Volume 4.

1. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses*. Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 27
FREEWAY WEAVING: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. ALTERNATIVE TOOL EXAMPLES FOR FREEWAY WEAVING SEGMENTS

Chapter 12, Freeway Weaving Segments, described a methodology for analyzing freeway weaving segments to estimate their capacity, speed, and density as a function of traffic demand and geometric configuration. Supplemental problems involving the use of alternative tools for freeway weaving sections to address limitations of the Chapter 12 methodology are presented here. All of these examples are based on Example Problem 1 in Chapter 12. The weaving segment configuration shown in that problem is repeated here as Exhibit 27-1 for convenience.

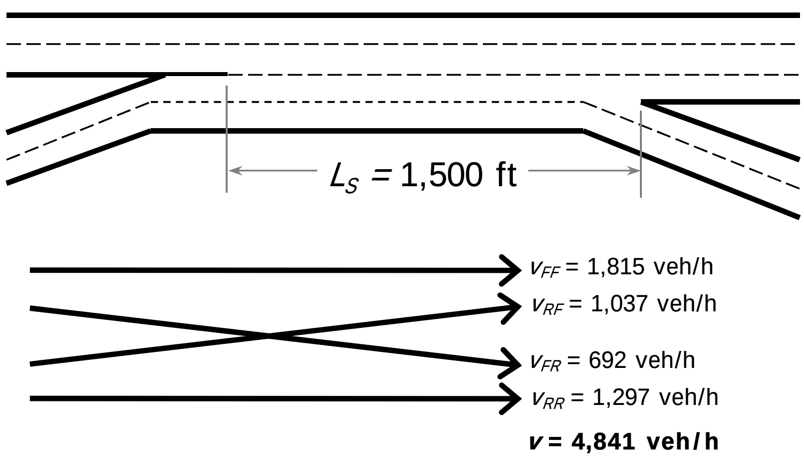


Exhibit 27-1
Major Weaving Segment for Example Problem 1

Three questions are addressed by using a typical link-node-based microscopic traffic simulation tool:

1. Can weaving segment capacity be estimated realistically by simulation by varying the demand volumes up to and beyond capacity?
2. How does demand affect performance in terms of speed and density in the weaving segment, on the basis of the default model parameters for vehicle and behavioral characteristics?
3. How would the queue backup from a signal at the end of the off-ramp affect weaving operation?

The first step is to identify the link–node structure, as shown in Exhibit 27-2.

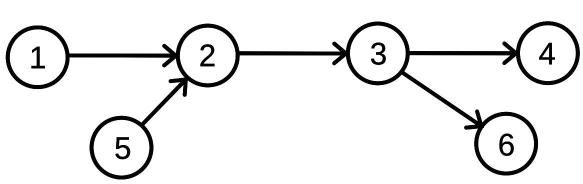


Exhibit 27-2
Link–Node Structure for the Simulated Weaving Segment

The next step is to develop input data for various demand levels. Several demand levels ranging from 80% to 180% of the original volumes were analyzed by simulation. The demand data, adjusted for a peak hour factor of 0.91, are given in Exhibit 27-3.

Exhibit 27-3
Input Data for Various Demand Levels

Percent of specified demand	80	100	120	140	160	180
Freeway-to-freeway demand, V_{FF}	1,596	1,995	2,393	2,792	3,191	3,590
Ramp-to-freeway demand, V_{RF}	912	1,140	1,367	1,595	1,823	2,051
Freeway-to-ramp demand, V_{FR}	608	760	913	1,065	1,217	1,369
Ramp-to-ramp demand, V_{RR}	1,140	1,425	1,710	1,995	2,280	2,565
Total demand	4,256	5,320	6,384	7,448	8,512	9,576
Total freeway entry	2,204	2,755	3,306	3,857	4,408	4,959
Total freeway exit	2,507	3,134	3,761	4,388	5,015	5,641
Total ramp entry	2,052	2,565	3,078	3,591	4,104	4,617
Total ramp exit	1,749	2,186	2,623	3,060	3,497	3,934

Thirty simulation runs were made for each demand level. The results are discussed in the following sections. The need to determine performance measures from an analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, the examples presented in this chapter have applied existing versions of alternative tools and therefore do not reflect the trajectory-based measures described in Chapter 7.

DETERMINING THE WEAVING SEGMENT CAPACITY

Simulation tools do not produce capacity estimates directly. The traditional way to estimate the capacity of a given system element is to overload it and determine the maximum throughput under the overloaded conditions. Care must be taken in this process because a severe overload can reduce the throughput by introducing self-aggravating phenomena upstream of the output point.

Exhibit 27-4 shows the relationship between demand volume and throughput, represented by the output of the weaving segment. As expected, throughput tracks demand precisely up to the point where no more vehicles can be accommodated. After that point it levels off and reaches a constant value that indicates the capacity of the segment. In this case, capacity was reached at approximately the same value as the HCM estimate. However, this degree of agreement between the two estimation techniques should not be expected as a general rule because of differences in the treatment of vehicle and geometric characteristics.

On the basis of observation, it is reasonable to conclude that the capacity of this weaving segment can be determined by overloading the facility and that the results are in general agreement with those of the HCM. In comparing capacity estimates, it must be remembered that the HCM expresses results in passenger car equivalent vehicles, while simulation tools express results in actual vehicles. The results will diverge as the proportion of trucks increases.

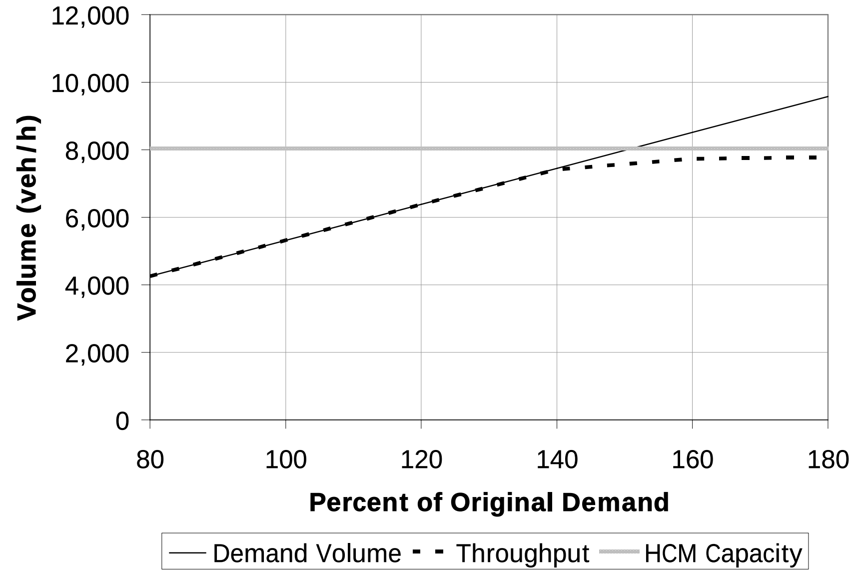


Exhibit 27-4
Determining the Capacity of a Weaving Segment by Simulation

LIVE GRAPH
Click here to view

EFFECT OF DEMAND ON PERFORMANCE

Exhibit 27-5 shows the effect of demand on density and speed. Density increases with demand volume up to the segment capacity and then levels off at a constant value of approximately 75 veh/ln/mi, which represents very dense conditions. The speed remains close to the free-flow speed at lower demand volumes. It then drops in a more or less linear fashion and eventually levels off when capacity is reached. The minimum speed is approximately 26 mi/h.

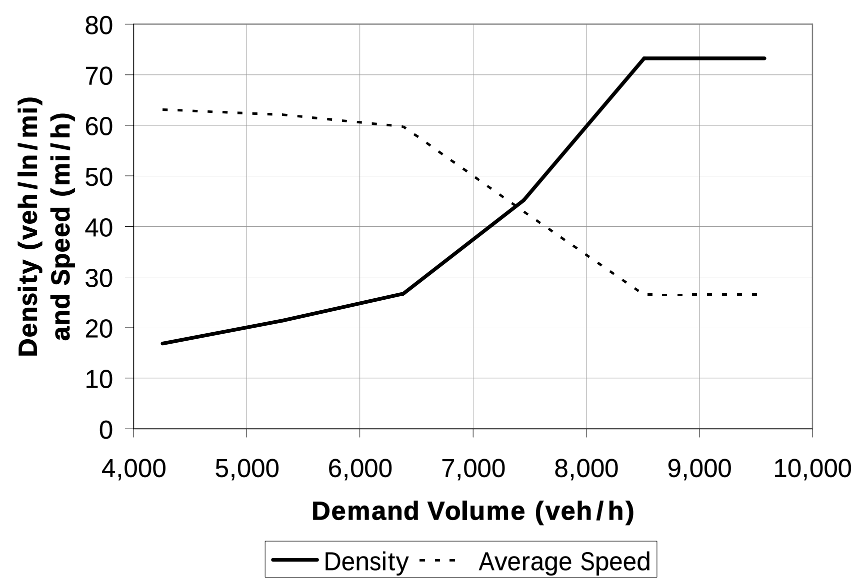


Exhibit 27-5
Simulated Effect of Demand Volume on Weaving Segment Capacity and Speed

LIVE GRAPH
Click here to view

At the originally specified demand volume level of 5,320 veh/h (peak hour adjusted), the estimated speed was 62.0 mi/h and the density was 21.4 veh/ln/mi. The corresponding values from simulation were 53.1 mi/h and 26.3 pc/ln/mi. Because of differences in definition, these results are not easy to compare. These differences illustrate the pitfalls of applying level of service (LOS) thresholds to directly simulated density to determine the segment LOS.

The densities produced when demand exceeded capacity were greater than 70 veh/ln/mi. This level of density is usually associated with queues that back up from downstream bottlenecks; however, in this case, no such bottlenecks were present. Inspection of the animated graphics suggests that the increase in density within the weaving segment is caused by vehicles that are not able to get into the required lane for their chosen exit. Some vehicles were forced to stop and wait for a lane-changing opportunity, and the reduction in average speed produced a corresponding increase in the average density.

For purposes of illustration, this example focuses on a single link containing the weaving segment. The overloading of demand prevented all of the vehicles from entering the link and would have increased the delay substantially if the vehicles denied entry were considered. For this reason, the delay measures from the simulation were not included in this discussion.

EFFECT OF QUEUE BACKUP FROM A DOWNSTREAM SIGNAL ON THE EXIT RAMP

The operation of a weaving segment may be expected to deteriorate when congestion on the exit ramp causes a queue to back up into the weaving segment. This condition was one of the stated limitations of the methodology in Chapter 12, Freeway Weaving Segments.

Signal Operation

To create this condition, a pretimed signal with a slightly oversaturated operation is added 700 ft from the exit point. The operating parameters for the signal are given in Exhibit 27-6. Note that the right-turn capacity estimated by the Chapter 18, Signalized Intersections, procedure is slightly lower than the left-turn capacity because of the adjustment factors applied to turns by that procedure.

Exhibit 27-6
Exit Ramp Signal Operating Parameters

Cycle length	150 s
Green interval	95 s
Yellow interval	4 s
All-red clearance	1 s
Saturation flow rate	1,800 veh/hg/ln
g/C ratio	0.633
Left-turn movement	
• Lanes	1
• Capacity (by HCM Chapter 18)	1,083 veh/h
Right-turn movement	
• Lanes	1
• Capacity (by HCM Chapter 18)	969 veh/h
Link capacity (by HCM Chapter 18)	2,052 veh/h

Capacity Calibration

To ensure that the simulation model is properly calibrated to the HCM, the simulation tool's operating parameters for the link were modified by trial and error to match the HCM estimate of the link capacity by overloading the link to determine its throughput. With a start-up lost time of 2.0 s and a steady-state headway of 1.8 s/veh, the simulated capacity for the link was 2,040 veh/h, which compares well with the HCM's estimate of 2,052 veh/h.

Results with the Specified Demand

An initial run with the demand levels specified in the original example problem indicated severe problems on the freeway caused by the backup of vehicles from the signal. Two adverse conditions are observed in the graphics capture shown in Exhibit 27-7:

1. Some vehicles in the freeway mainline through lanes were unable to access the auxiliary lane for the exit ramp because of blockage in the lane.
2. The resulting use of the exit ramp lanes prevented the signal operation from reaching its full capacity. This caused a self-aggravating condition in which the queue backed up farther onto the freeway.

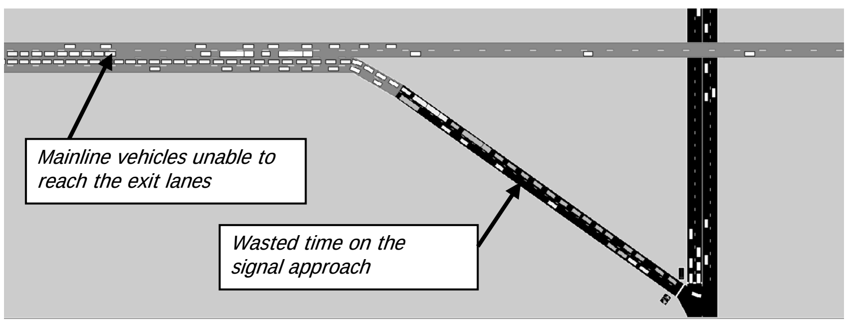


Exhibit 27-7
Deterioration of Weaving Segment Operation due to Queue Backup from a Traffic Signal

A reasonable conclusion is that the weaving segment would not operate properly at the specified demand levels. The logical solution to the problem would be to improve signal capacity. To support a recommendation for such an improvement, varying the demand levels to gain further insight into the operation might be desirable. Since it has already been discovered that the specified demand is too high, the original levels of 80% to 180% of the specified demand are clearly inappropriate. The new demand range will therefore be reduced to a level of 80% to 105%.

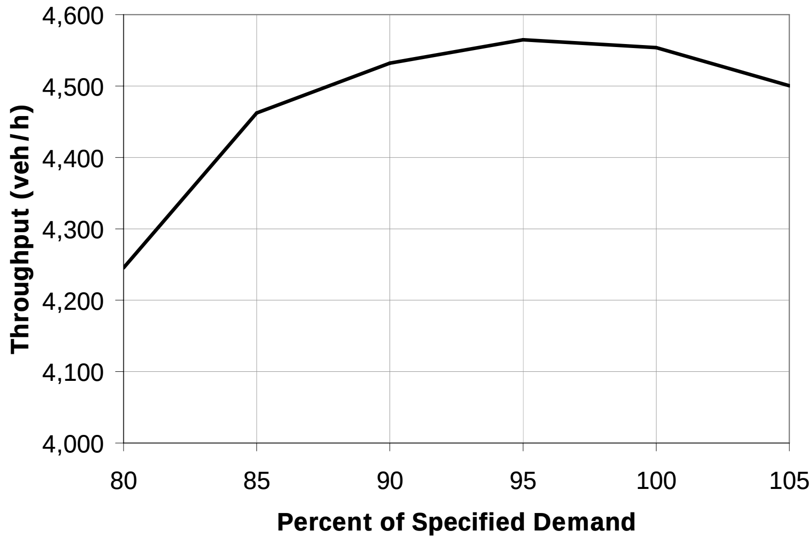
Effect of Reducing Demand on Throughput

Exhibit 27-8 illustrates the self-aggravating effect of too much demand. Throughput is generally expected to increase with demand up to the capacity of the facility and to level off at that point. Notice that the anticipated relationship was observed without the signal, as was shown in Exhibit 27-4.

When the signal was added, the situation changed significantly. The throughput peaked at about 95% of the specified demand and declined noticeably as more vehicles were allowed to enter the freeway. Another useful observation is that the peak throughput of approximately 4,560 veh/h is considerably below the estimated capacity of nearly 8,000 veh/h.

Exhibit 27-8
Effect of Demand on Weaving Segment Throughput with Exit Ramp Backup

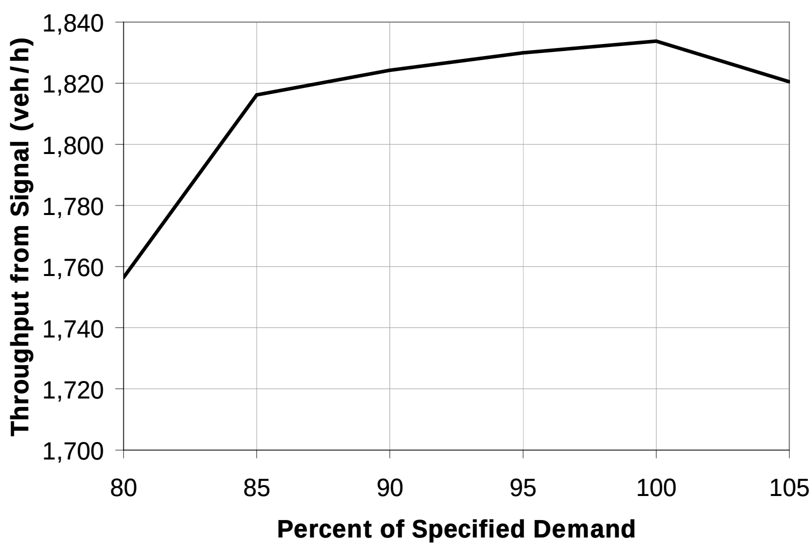
LIVE GRAPH
Click here to view



The same phenomenon is observed on the exit ramp approach to the signal, as shown in Exhibit 27-9. The throughput declined with added demand after reaching its peak value of about 1,835 veh/h. Note that the peak throughput is also well below the capacity of 2,040 to 2,050 veh/h estimated by both the HCM and the simulation tool in the absence of upstream congestion.

Exhibit 27-9
Effect of Demand on Exit Ramp Throughput with Signal Queuing

LIVE GRAPH
Click here to view



This example illustrates the potential benefits of using simulation tools to address conditions that are beyond the scope of the HCM methodology. It also points out the need to consider conditions outside of the facility under study in making a performance assessment. Finally, it demonstrates that care must be taken in estimating the capacity of a facility through an arbitrary amount of demand overload.

CHAPTER 28
FREEWAY MERGES AND DIVERGES: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. ALTERNATIVE TOOL EXAMPLES FOR FREEWAY RAMPS

Chapter 13, Freeway Merge and Diverge Segments, described a methodology for analyzing ramps and ramp junctions to estimate capacity, speed, and density as a function of traffic demand and geometric configuration. This chapter includes two supplemental problems that examine situations that are beyond the scope of the Chapter 13 methodology. A typical microsimulation-based tool is used for this purpose, and the simulation results are compared, where appropriate, with those of the *Highway Capacity Manual* (HCM).

Both problems are based on Chapter 13's Example Problem 3, which analyzes an eight-lane freeway segment with an entrance and an exit ramp. The first problem evaluates the effects of the addition of ramp metering, while the second evaluates the impacts of converting the leftmost lane of the mainline into a high-occupancy vehicle (HOV) lane.

The need to determine performance measures based on the analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, the examples presented in this chapter have applied existing versions of alternative tools and therefore do not reflect the trajectory-based measures described in Chapter 7.

For purposes of illustration, the default calibration parameters of the simulation tool (e.g., lane-changing behavioral characteristics) were applied to these examples. However, most simulation tools offer the ability to adjust these parameters. The parameter values can have a significant effect on the results, especially when the operation is close to full saturation.

PROBLEM 1: RAMP-METERING EFFECTS

This problem analyzes the impacts of ramp metering along the segment. The HCM procedure for ramp-merge junctions cannot estimate the impacts of ramp metering. These impacts can be approximated to some extent by not allowing the ramp demand to exceed the ramp-metering rate. To address ramp metering at a more detailed level, a typical microsimulation tool was used to evaluate the impacts of ramp metering on the density and capacity of the merge.

The subject segment consists of an on-ramp followed by an off-ramp, separated by 1,300 ft. The upstream segment is 1 mi long. Each simulation run was for 1 full hour. It was assumed that the mainline demand was 6,111 veh/h and that the ramp demand was 444 veh/h. The ramp metering is clock-time based (i.e., the metering rate does not change as a function of the mainline demand).

Experiments were conducted to obtain the density and capacity of the subject segment as a function of the ramp-metering rate. The queue length upstream of the ramp meter was also obtained as a function of the ramp-metering rate. Exhibit 28-1 provides a graphics capture of the simulated site.

Exhibit 28-1
Graphics Capture of the Ramp Merge with Ramp Metering

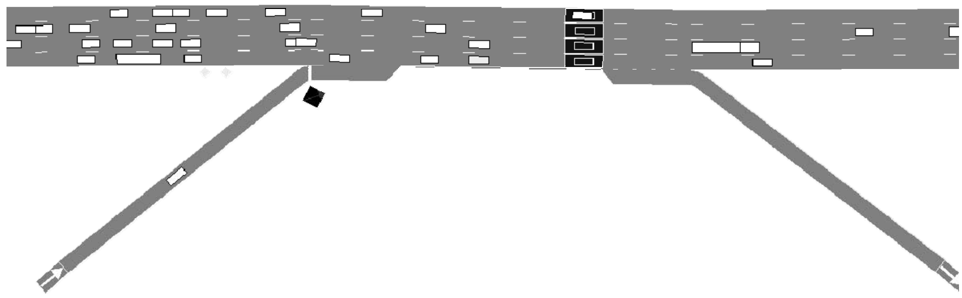


Exhibit 28-2 provides the density of the segment between the on-ramp and the off-ramp as a function of the ramp-metering rate (or discharge headway from the on-ramp). As shown, the density is not much affected by the ramp-metering rate. As expected, the density of Lane 1 (the rightmost lane) is the highest, while the density in Lane 4 is the lowest.

Exhibit 28-2
Density as a Function of Ramp-Metering Headways

LIVE GRAPH
Click here to view

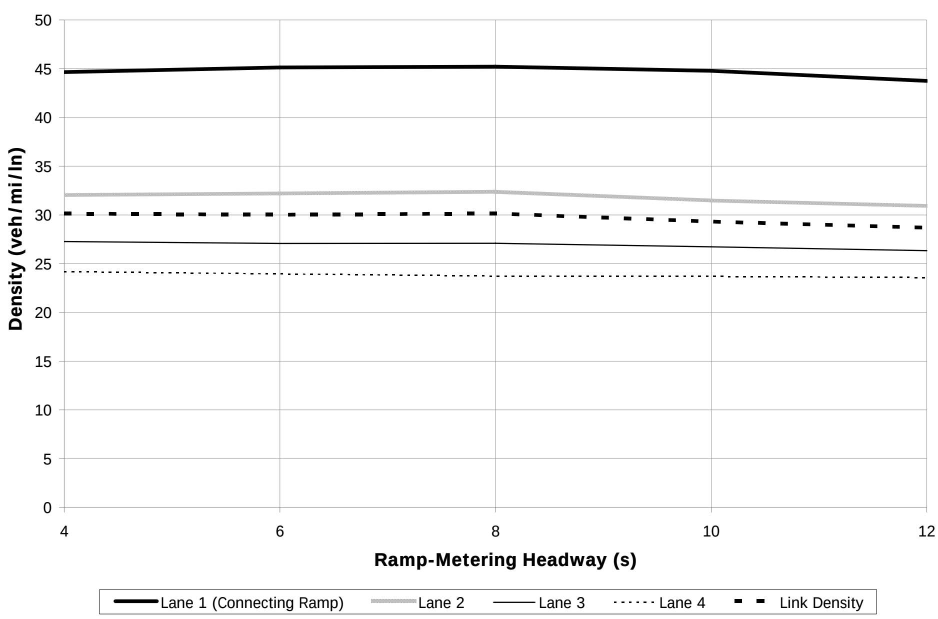


Exhibit 28-3 provides capacity as a function of the ramp-metering headway and the estimated maximum throughput when no ramp metering is implemented. As shown, the simulation model predicts that capacity is higher when ramp metering is implemented.

Exhibit 28-3
Capacity at a Ramp Junction as a Function of Ramp-Metering Headways

LIVE GRAPH
Click here to view

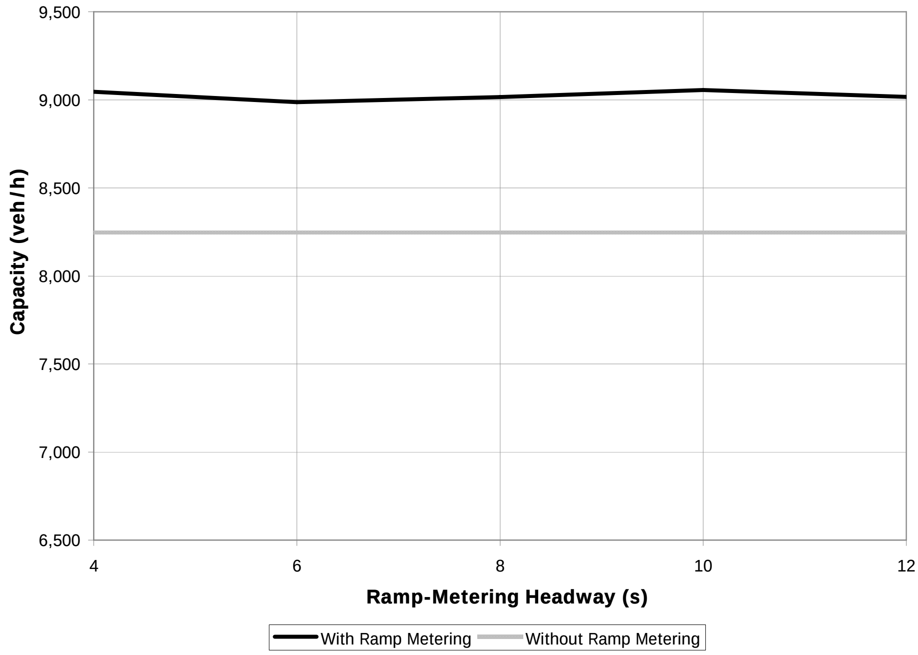
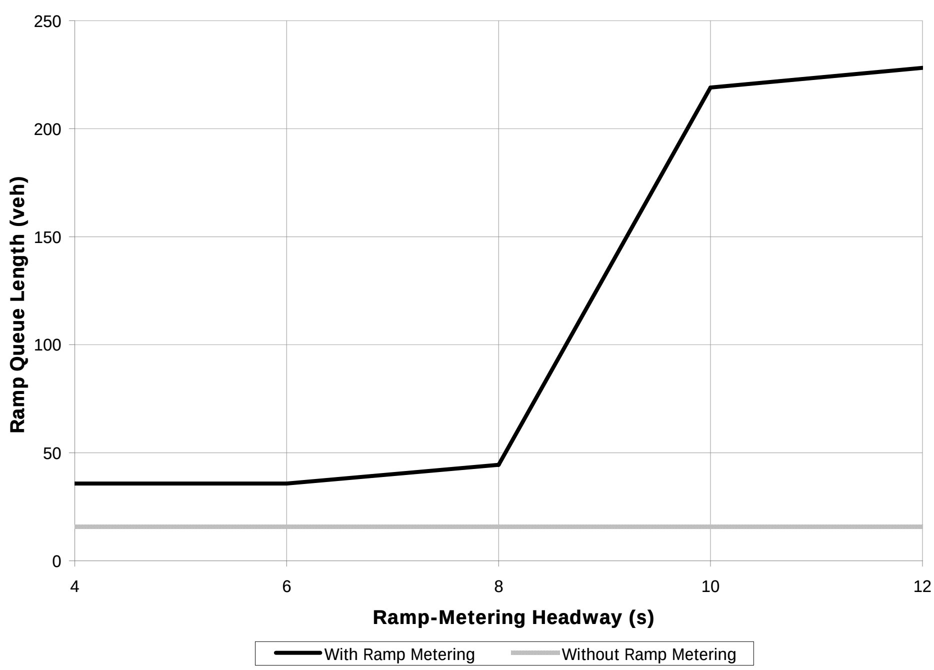


Exhibit 28-4 provides the queue length expected on the ramp, as a function of ramp metering and when no ramp metering is implemented. As expected, the queue length is somewhat higher when ramp metering is implemented, and it increases dramatically when the ramp-metering rate exceeds 8 s/veh.

Exhibit 28-4
Queue Length on the Ramp as a Function of Ramp-Metering Headways

LIVE GRAPH
Click here to view



As indicated above, the effects of ramp metering cannot be evaluated with the HCM. The freeway facilities methodology (HCM Chapter 10) can handle changes in segment capacity; however, other tools are required to estimate what the maximum throughput would be under various types of ramp-metering

algorithms and rates. Also, the HCM cannot estimate the queue length on the on-ramp as a function of ramp metering. An analytical method could be developed to estimate queue length as a function of demand and service rate at the meter.

PROBLEM 2: CONVERSION OF LEFTMOST LANE TO AN HOV LANE

This problem is also based on Chapter 13's example problem. It evaluates operating conditions when the leftmost lane of the mainline is converted into an HOV lane. Exhibit 28-5 provides a graphics capture of the segment.

Exhibit 28-5
Graphics Capture of the Segment with an HOV Lane

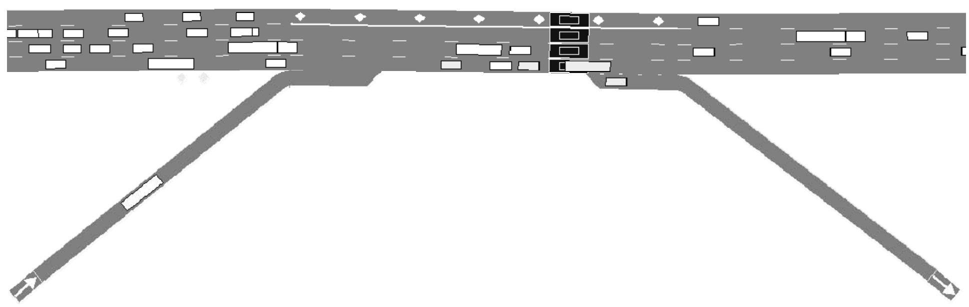
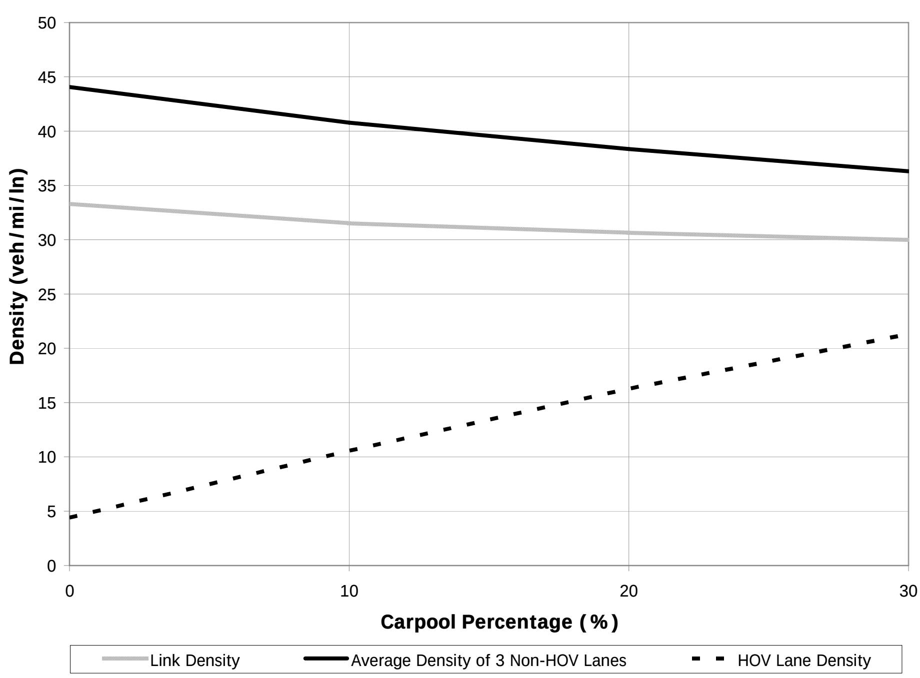


Exhibit 28-6 and Exhibit 28-7 show the density and capacity of the ramp junction as a function of the percentage of carpools. As shown, when the percentage of carpools increases, the density of the HOV lane and the overall link capacity increase. This occurs because for the range of values tested here, the utilization of the HOV increases, thus improving the overall link performance.

Exhibit 28-6
Density of a Ramp Junction as a Function of the Carpool Percentage

LIVE GRAPH
Click here to view



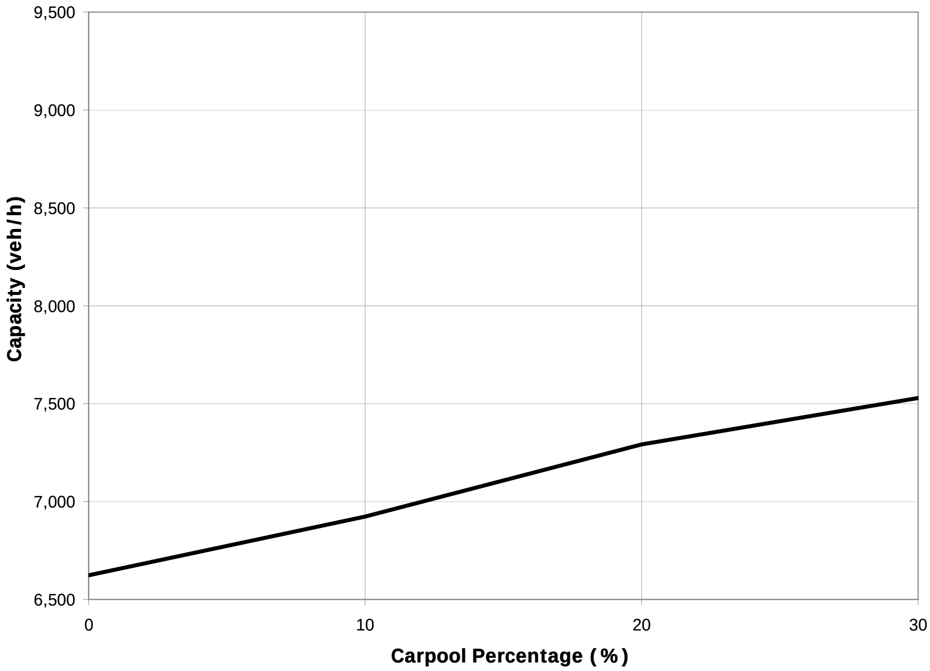


Exhibit 28-7
Capacity of a Ramp Junction as a Function of the Carpool Percentage

LIVE GRAPH
Click here to view

Exhibit 28-8 presents the density as a function of HOV violators, while Exhibit 28-9 presents the corresponding capacity. These two graphs assume that there are 10% carpools in the traffic stream. As shown, density generally decreases while capacity increases as the percentage of HOV violators increases. The reason is that under this scenario, the facility is more efficiently utilized as violations increase with general traffic using the HOV lane.

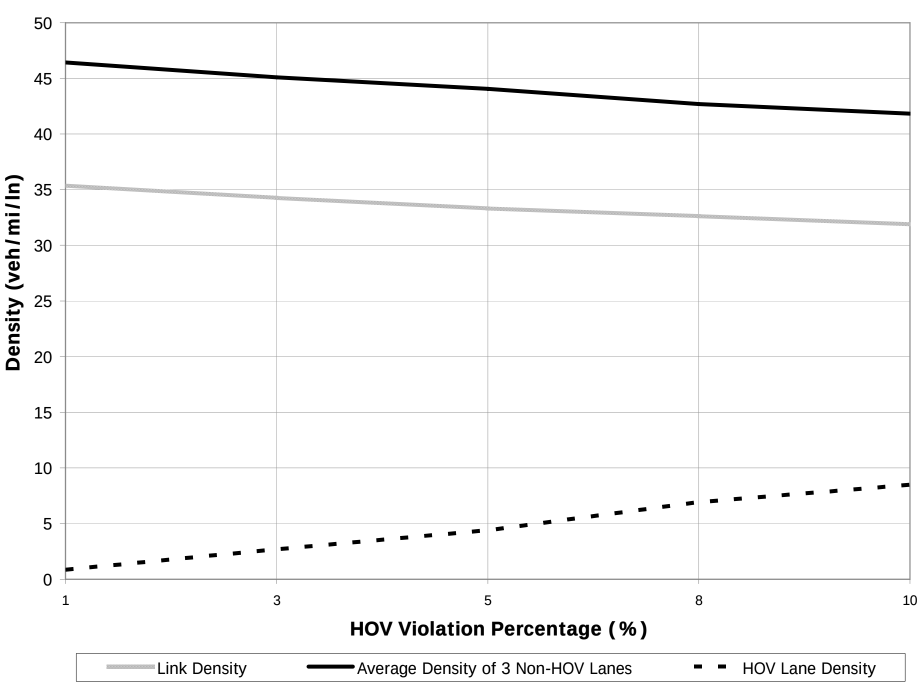


Exhibit 28-8
Density of a Ramp Junction as a Function of the HOV Violation Percentage

Exhibit 28-9
Capacity of a Ramp Junction as a Function of the HOV Violation Percentage

LIVE GRAPH
Click here to view

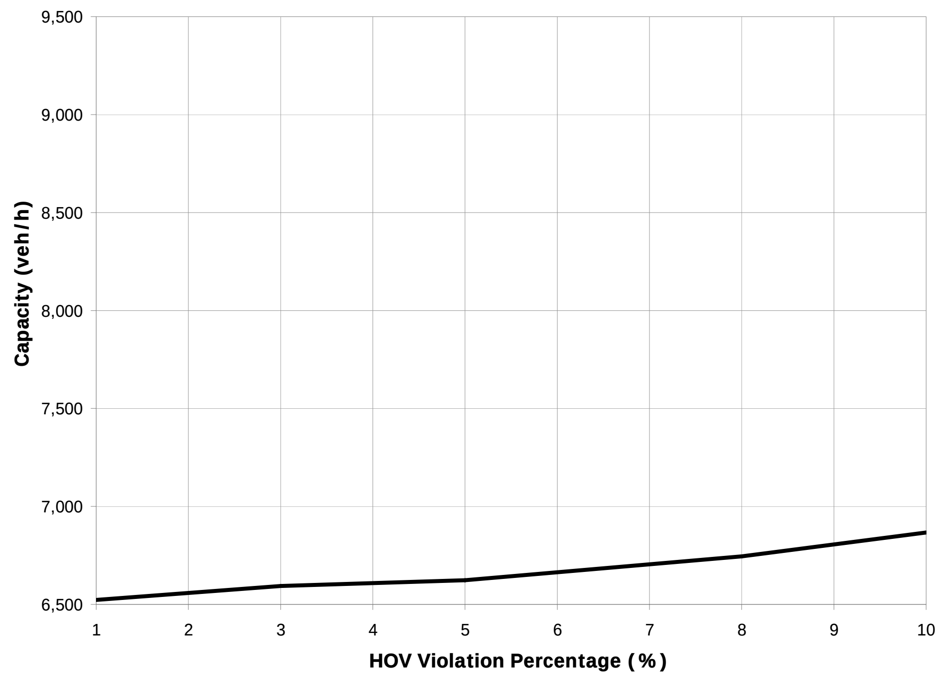
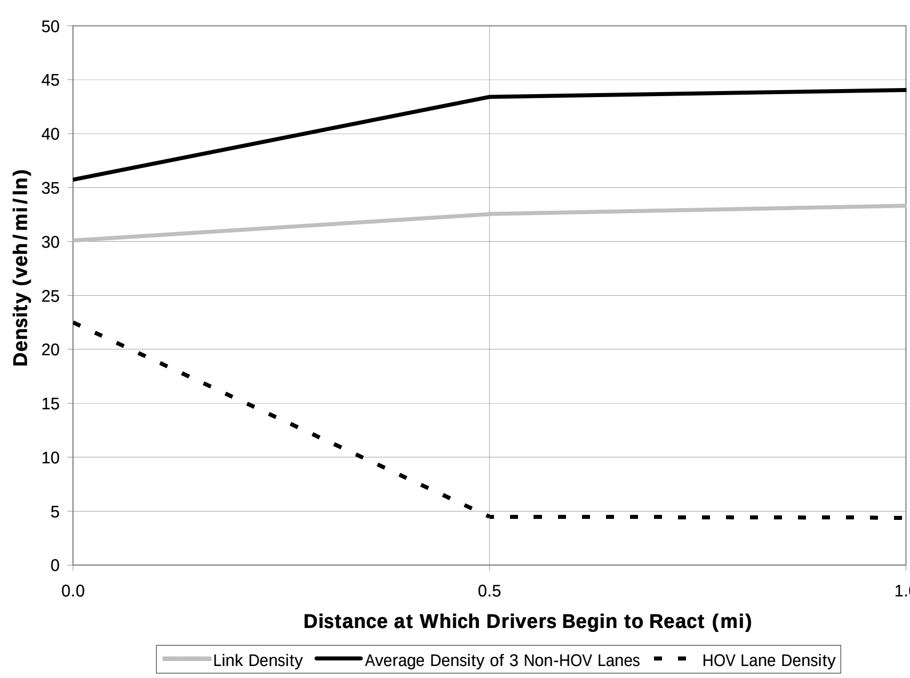


Exhibit 28-10 and Exhibit 28-11 present the density and capacity of the ramp junction as a function of the distance at which drivers begin to react to the presence of the HOV lane (i.e., the distance to the regulatory sign). As shown, the longer that distance, the lower the density of the HOV lane, and the higher the density in the other lanes. The reason is that under this scenario the percentage of carpools is relatively low (10%). When the HOV lane begins, non-HOVs congregate in the remaining lanes. Capacity is reduced as the distance at which drivers begin to react increases, because the HOV lane is not utilized as much when drivers are given early warning to switch lanes.

Exhibit 28-10
Density of a Ramp Junction as a Function of the Distance at Which Drivers Begin to React

LIVE GRAPH
Click here to view



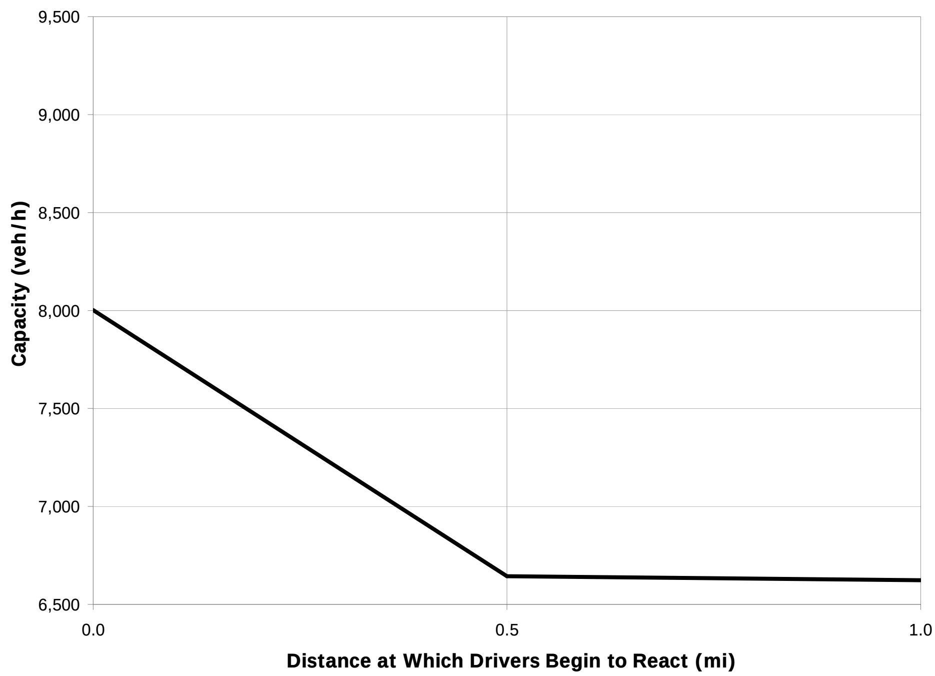


Exhibit 28-11
Capacity of a Ramp Junction as a Function of the Distance at Which Drivers Begin to React

LIVE GRAPH
Click here to view

Exhibit 28-12 and Exhibit 28-13 present the density and capacity of the ramp junction as a function of the percentage of HOV usage. As expected, when usage of the HOV lane increases, the density of the HOV lane and the overall link capacity increase.

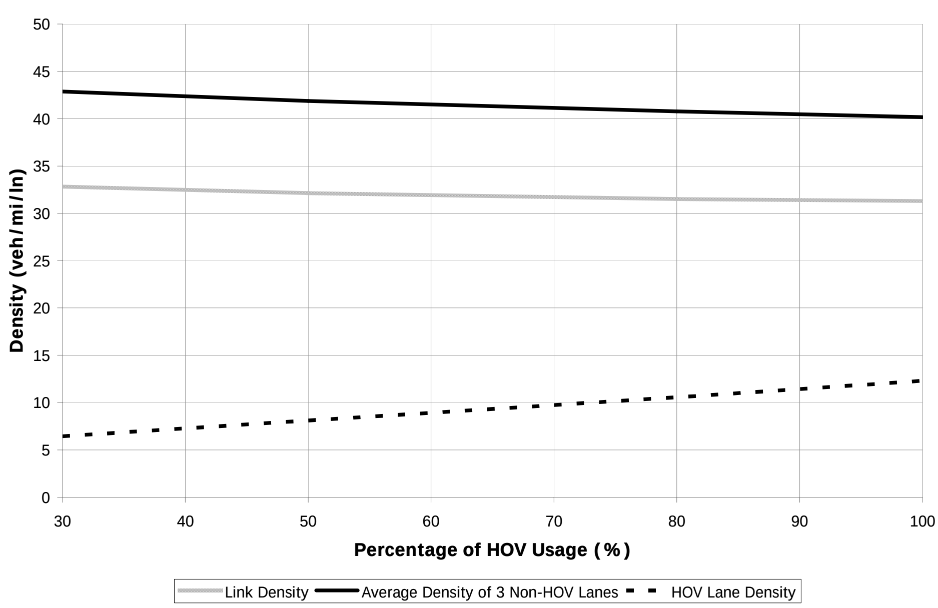
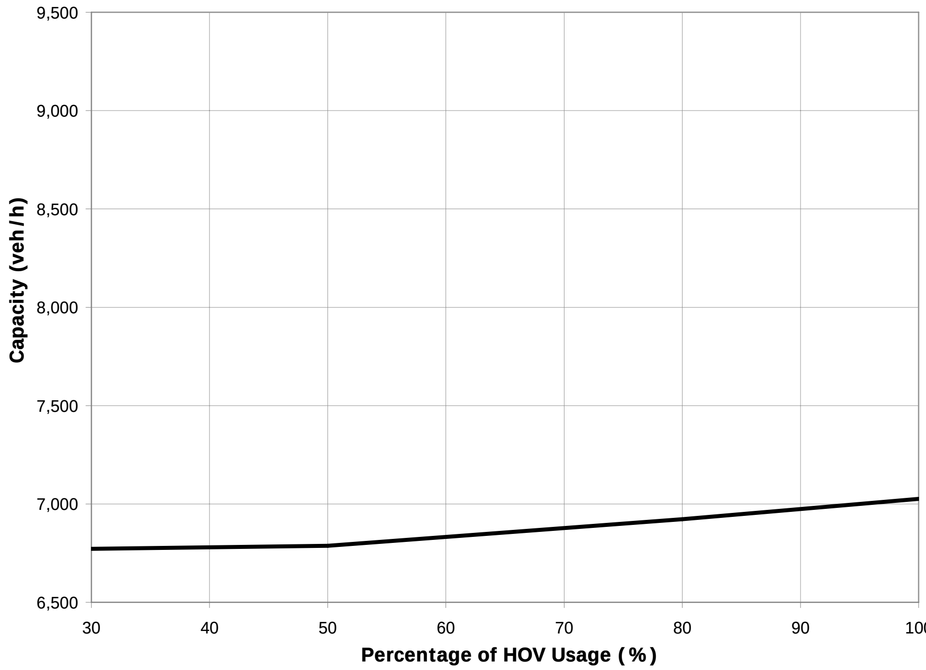


Exhibit 28-12
Density of a Ramp Junction as a Function of the Percentage of HOV Usage

LIVE GRAPH
Click here to view

Exhibit 28-13
Capacity of a Ramp Junction as a Function of the Percentage of HOV Usage

LIVE GRAPH
Click here to view



The type of analysis presented in this example cannot be conducted with the HCM, since the method does not estimate the HOV lane density separately. Variables such as the impact of the distance of the HOV regulatory sign cannot be evaluated, since they pertain to driver behavior attributes and their impact on density and capacity. The impact of the percentage of carpools and the percentage of violators could perhaps be estimated with appropriate modifications of the existing HCM method.

CHAPTER 29
URBAN STREET FACILITIES: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 16, Urban Street Facilities, presented a methodology for combining the performance measures from each segment on an urban street in a manner that represents the operation of the facility as a whole. The stated limitations for each type of segment apply equally to the analysis of the facilities that they make up. In addition, the Chapter 16 procedures do not recognize interactions between segments that could occur, for example, when a queue crosses segment boundaries or when the operation within a segment or at a segment boundary disturbs the progressive movement of traffic along a route.

Several other chapters present supplemental examples covering the use of alternative tools to deal with individual segment limitations such as queue spillover, interaction between segments, and certain types of self-aggravating phenomena:

- Chapter 24, Concepts: Supplemental, demonstrates the use of individual vehicle trajectory analysis to examine cyclical queuing characteristics and to assess queue spillover into an upstream segment.
- Chapter 27, Freeway Weaving: Supplemental, presents a simulation example that demonstrates the detrimental effect of queue backup from an exit ramp signal on the operation of a freeway weaving section.
- Chapter 31, Signalized Intersections: Supplemental, presents simulation examples that demonstrate the effect of storage bay overflow, right-turn-on-red operation, short through lanes, and closely spaced intersections.
- Chapter 34, Interchange Ramp Terminals: Supplemental, presents a simulation example that demonstrates the effect of ramp metering signals on the operation of a diamond interchange. Another simulation example examines the effect of the diamond interchange on the operation of a nearby intersection under two-way stop control.

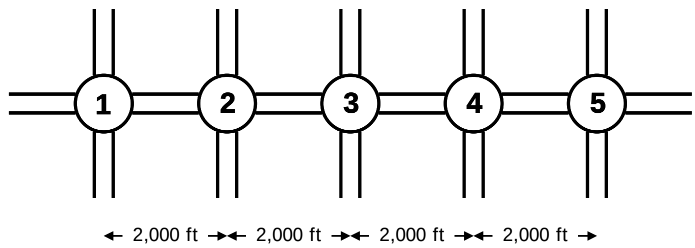
This chapter presents a few supplemental examples using alternative automobile traffic analysis tools to deal specifically with the limitations of the Chapter 16 procedures and to capitalize on the additional features of alternative tools. Both deterministic and stochastic tools are illustrated. The emphasis is exclusively on the automobile mode because alternative tools are applied more frequently to deal with automobile traffic.

The need to determine performance measures from an analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results, and Chapter 24, Concepts: Supplemental. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, most of the examples presented in this chapter have applied existing versions of alternative tools and therefore do not reflect the proposed trajectory-based measures.

2. BASIC EXAMPLE PROBLEM CONFIGURATION

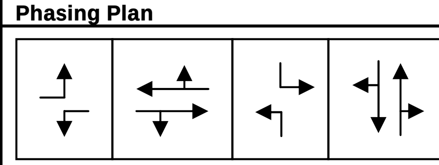
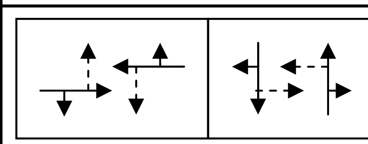
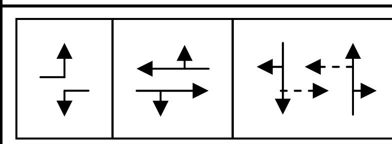
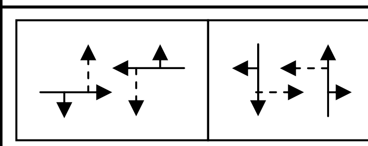
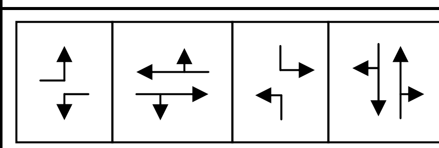
The base configuration for these examples is shown in Exhibit 29-1. Five signalized intersections are included with a spacing of 2,000 ft between the upstream stop lines of each intersection. Each intersection has the same layout, with two lanes for through and right-turn movements and one 150-ft-long left-turn bay.

Exhibit 29-1
Base Configuration for the Examples



The phasing and demand flow rates for each intersection are shown in Exhibit 29-2. Leading protected phases are provided for all protected left turns. Intersections 1 and 5 have protected phases for all left turns. Intersections 2 and 4 have only permitted left turns. Intersection 3 has protected left turns on the major street and permitted left turns on the minor street.

Exhibit 29-2
Demand Flow Rates and Phasing Plan for Each Intersection

Int. No.	Movement	Peak 15-min Adjusted Demand: Left	Through	Right	Phasing Plan
1	Major st.	120	800	80	
	Minor st.	120	600	80	
2	Major st.	80	800	120	
	Minor st.	80	600	120	
3	Major st.	120	800	80	
	Minor st.	80	600	120	
4	Major st.	80	800	120	
	Minor st.	80	600	120	
5	Major st.	120	800	80	
	Minor st.	120	600	80	

To simplify the discussion, the examples will focus on design and analysis features that are beyond the stated limitations of the urban street analysis procedures contained in Chapters 16 through 21. For example, pretimed control will be assumed here because the ability to deal with traffic-actuated control is not a limitation of the Chapter 18 signalized intersection analysis methodology. For the same reason, the analysis of complex phasing schemes that fall within the scope of the Chapter 18 procedures (e.g., protected-permitted phasing) will be avoided. Parameters that influence the saturation flow rate (e.g., trucks, grade, lane width, parking) will not be considered here because they are accommodated in other chapters.

A symmetrical demand volume pattern will be used to facilitate interpretation of results. The demand volumes are assumed to be peak-hour adjusted. Fixed yellow-change and all-red clearance intervals of 4 s and 1 s, respectively, will be assigned to all phases. Through-traffic phases and protected left-turn phases will be assigned minimum green times of 10 s and 8 s, respectively.

3. SIGNAL TIMING PLAN DESIGN

The procedures presented throughout the *Highway Capacity Manual* (HCM) were developed to determine the performance of a roadway segment under specific conditions. In simple cases the procedures may be applied in reverse for design purposes (e.g., determining the number of required lanes). In more complex situations requiring optimization of design parameters, the procedures must be applied iteratively within an external software structure. Some alternative tools provide this type of optimization structure and therefore offer a valuable extension of the HCM's methodology. The extent of HCM compatibility in the analysis methodology varies among tools.

Two deterministic optimization tools will be applied in this section to illustrate how they can be used to produce the signal timing parameters required by the procedures of Chapters 17 and 18. This discussion is not intended as a comprehensive tutorial on signal timing plan design (STPD). A more detailed treatment of this subject is available (*1*), which serves as a comprehensive guide to traffic signal timing and includes a discussion of the use of deterministic optimization tools. It represents a synthesis of traffic signal timing concepts and their application and focuses on the use of detection, related timing parameters, and effects on users at the intersection.

DETERMINISTIC STPD TOOLS

Several deterministic STPD tools are available commercially. Each represents a comprehensive package with its own computational and interface features designed to provide insight into operational details and to promote user productivity in the development of signal timing plans. A typical STPD tool configuration is illustrated in Exhibit 29-3. The following elements are included in the configuration:

- The computational model, which performs the design, optimization, and analysis functions. Two components are included in the computational model. The first component computes performance measures on the basis of specified input data and operating parameters. The second contains the optimization routines that seek a combination of operating parameters that will produce the best performance.
- The data input editor, which organizes and facilitates the entry of traffic data and operating parameters to be supplied to the computational model. The data input editor establishes the "look and feel" of each tool. The details vary considerably among tools. For example, some tools offer the ability to compute saturation flow rates internally by using procedures similar to those prescribed in Chapter 18, Signalized Intersections.
- Import/export features, which facilitate communication of data sets between other applications and devices. These features are intended to enhance the productivity of each tool.

- Direct links to other applications, such as microscopic simulation tools and fully HCM-compliant software.
- Graphic displays, which provide insight into time–space relationships, queuing, and platoon propagation.

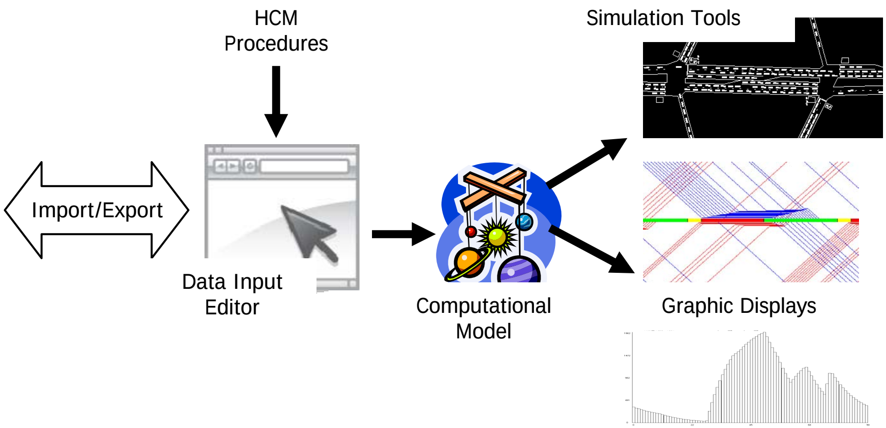


Exhibit 29-3
Elements of a Typical Signal Timing Design Tool

The urban streets analysis procedures presented in the HCM deal with the operation of a single artery as a set of interconnected segments. Most of the commonly used STPD tools are configured to accommodate traffic control networks involving multiple intersecting routes. To simplify the discussion, the example presented here is limited to a single arterial route that will be analyzed as a system.

Two widely used STPD tools will be applied to this example to illustrate their features and to show how they can be used to supplement the urban street facilities analysis procedures prescribed in this manual. Both tools are commercially available software products. More information about these tools can be found elsewhere (*2*, *3*). The discussion in this section deals with the combination of features available from both tools without reference to a specific tool.

PERFORMANCE MEASURES

Both STPD tools deal with performance measures that are computed by the procedures prescribed in this manual in addition to performance measures that are beyond the scope of those procedures. The performance measures covered in Chapters 16 and 17 include delay, stops, average speed, and queue length. The discussion of those measures in this section will focus on their use in STPD and not on comparison of the values computed by different methods.

Several other measures beyond the scope of the HCM procedures are commonly associated with signal timing plan design and evaluation. The following measures are derived from analysis of travel characteristics, including stops, delay, and queuing:

- *Fuel consumption* (gal/h), the amount of fuel consumed because of vehicle miles traveled, stops, and delay, as computed by a model specific to each tool;

- *Operating cost* ($/h), the total cost of operation of all vehicles as computed by a model specific to each tool; and
- *Time jammed*, the percentage of time that the queue on a link has backed up beyond the link limit.

STPD tools also deal with a set of performance measures related to the quality of progression between intersections. These measures, all of which are outside of the HCM scope, have been defined in the literature or by developers of specific tools as follows:

- *Bandwidth* is defined by the number of seconds during which vehicles traveling at the design speed will be able to progress through a set of intersections. *Link bandwidth* is the width of the progression band (in seconds) passing between adjacent intersections that define the link. *Arterial bandwidth* is the width of the progression band that travels the entire length of the arterial route.
- *Progression efficiency* is the ratio of the arterial bandwidth to the cycle length. It thus represents the proportion of the cycle that contains the arterial progression band. Suggested upper limits for "poor," "fair," and "good" progression are 0.12, 0.24, and 0.36, respectively (*3*). Values above 0.36 are characterized as "great" progression.
- *Progression attainability* is the ratio of the arterial bandwidth to the shortest green time for arterial through traffic on the route. By definition, the arterial progression band cannot be greater than the shortest green time. Therefore, an attainability of 100% indicates that further improvement is only possible through the provision of additional green time. The need for fine-tuning is suggested for attainability values between 70% and 99%, with major changes needed for values below 70% (*3*).
- *Progression opportunities (PROS)* are a measure of arterial progression quality that recognizes progression bands that are continuous between two or more consecutive links but do not travel the full length of the arterial. The number of PROS observed by a driver at any point in time and space is defined by the number of intersections that lie ahead within the progression band. The concept is based on the premise that driver perception of progression quality increases with the number of consecutive links that can be traversed within the progression band. The measure is accumulated in a manner similar to the score in a game of bowling, where success in one frame is passed on to the next frame to increase the total score if the success continues. More detailed information on the computation of PROS is available elsewhere (*3*).
- *Interference* is expressed as the percentage of time that an arterial through vehicle entering a link on the green signal and traveling at the design speed will be stopped at the next signal. This measure is arguably an indication of poor perceived progression quality (*3*).
- *Dilemma zone vehicles* indicates the number of vehicles arriving on the yellow interval. As such, it offers a potential safety-related measure. The computational details are described elsewhere (*2*).

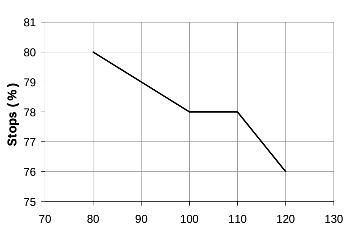
- *Coordinatability factor (CF)*: While it is not strictly a performance measure as defined in this manual, the CF is a measure of the desirability of coordinating two intersections on the basis of several factors including intersection spacing, speeds, and platoon formation. It is expressed as a relative value between 0 and 100. This measure is described in more detail elsewhere (*2*), where it is suggested that values above 80 indicate a definite need for coordination.

INITIAL TIMING PLAN DESIGN

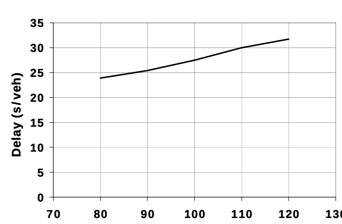
An initial timing plan design will first be performed by using one of the STPD tools. From the list of performance measures just discussed, fuel consumption will be chosen in this example as the performance measure for optimization. Other measures or combinations of measures could have been selected. No recommendation is implied in the selection of this particular measure. It serves this discussion because it supports an analysis of the trade-off between other measures such as stops and delay.

A cycle length within a specified range must be selected first. Minimum and maximum cycle lengths of 80 and 120 s, respectively, will be used. The cycle optimization results are presented in Exhibit 29-4, which shows the effect of the cycle length on delay, stops, and fuel consumption as computed by the STPD. While delay and stops move in opposite directions, their combined effect suggests that the minimum fuel consumption will be reached with an 80-s cycle. This is not surprising because it is generally recognized that the optimal cycle length for balanced progression is twice the link travel time at the design speed, which is 2 × 34 = 68 s for a 2,000-ft link at 40 mi/h. However, 68 s is below the minimum cycle length constraint. On the basis of these results, an 80-s cycle will be selected for optimization of the other timing plan parameters.

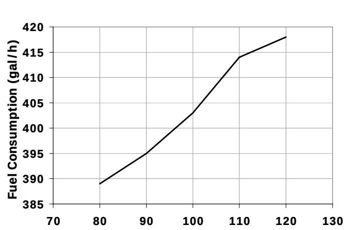
Exhibit 29-4
Cycle Length Optimization Results



(a) Stops Optimization



(b) Delay Optimization



(c) Fuel Consumption Optimization

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The split and offset optimization was carried out next. The resulting timing plan is shown in Exhibit 29-5. This table represents the initial timing plan to be investigated and refined.

Exhibit 29-5
Timing Plan Developed by Split and Offset Optimization

Intersection	Offset	Phase 1	Phase 2	Phase 3	Phase 4	Total
1	0	13	29	13	25	80
2	34	45	35			80
3	3	13	33	34		80
4	31	45	35			80
5	78	13	29	13	25	80

Notes: All times are in seconds.
Offsets are referenced to the first arterial through-traffic phase.

INITIAL TIMING PLAN PERFORMANCE

A summary of the performance measures for the initial timing plan is presented in Exhibit 29-6. Separate columns are included in this table for *route totals*, which include only the segments that make up the urban street facility as defined in Chapter 16, and *system totals,* which include the measures from the cross-street segments. Note that some of the performance measures reported in this table are also reported by the Chapter 16 procedures. While the STPD tool definitions and model structures are similar to the HCM (e.g., uniform and random components), no comparison of the values will be offered in this discussion because the focus is on the STPD and not on modeling differences.

Exhibit 29-6
Performance Measures for the Initial Timing Plan

Performance Measure	Units	System Totals	Route Totals
Total travel	veh-mi/h	4,927	3,063
Total travel time	veh-h/h	240	120
Uniform delay	veh-h/h	95	34
Random delay	veh-h/h	22	8
Total delay	veh-h/h	116	43
Average delay	s/veh	23.5	17.4
Passenger delay	p-h/h	140	51
Uniform stops	veh/h	12,893	5,576
Uniform stops	%	72	63
Random stops	veh/h	1,277	440
Random stops	%	7	5
Total stops	veh/h	14,171	6,016
Total stops	%	79	68
Links with d/c >1		0	0
Links with queue overflow		0	0
Time jammed	%	0	0
Period length	s	900	900
System speed	mi/h	20.5	25.6
Fuel consumption	gal/h	387	195
Operating cost	$/h	3,063	1,049

The initial timing plan design was based on minimizing fuel consumption as a performance measure. The signal progression characteristics of this design are also of interest. The progression characteristics will be examined in both numerical and graphics representations. The numbers are presented in Exhibit 29-7 and are based on the progression performance measures that were defined earlier. The interference values indicate the proportion of time that a vehicle entering a link in the progression band would be stopped at the next signal. The PROS are accumulated from progression bands that pass through some adjacent signals along the route. The low progression efficiency and attainability and PROS values suggest that this design, while optimal in some respects, would not produce a very favorable motorist perception of progression quality.

Performance Measure	Westbound	Eastbound	Average
Bandwidth efficiency	10%	5%	8%
Progression attainability	28%	14%	21%
Interference	9%	10%	
PROS	30%	28%	29%

Exhibit 29-7
Progression Quality Measures for the Initial Design

ADJUSTMENTS TO IMPROVE PROGRESSION QUALITY

Because of the low quality of progression, it is logical to revisit the initial design with the objective of maximizing progression quality instead of minimizing fuel consumption. The same cycle length range (80 to 120 s) was used for this purpose, and the runs were repeated with the objective of maximizing PROS. The maximum value of PROS was obtained with the same cycle length and phase times as the initial design. The progression performance measures associated with this timing plan are shown in Exhibit 29-8. These measures do not differ substantially from the initial design, nor do the offsets. The total PROS value increased from 29% to 30%, but the performance was somewhat better balanced by direction. Thus, there is not a large trade-off between the objectives of maximizing performance and maximizing progression quality in this case.

A combination of factors peculiar to this example has led to the conclusion that the signal timing parameters for optimizing performance and progression are basically the same. The symmetry of the layout and phasing created a situation in which fuel consumption could be minimized by favoring either direction at the expense of the other. The balanced design was favored by the PROS optimization because it offered a minimal numerical advantage (30% versus 29%). One of the main reasons why both design approaches chose the lowest acceptable cycle length is that, as pointed out previously, the theoretical optimum cycle length was below the lowest acceptable cycle length.

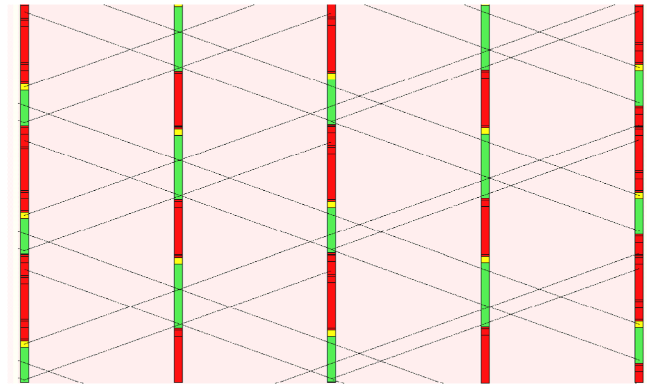
Performance Measure	Westbound	Eastbound	Average
Bandwidth efficiency	8%	8%	8%
Attainability	21%	21%	21%
Interference	9%	9%	
PROS	30%	30%	30%

Exhibit 29-8
Progression Quality Measures for the Improved Progression Design

TIME–SPACE DIAGRAMS

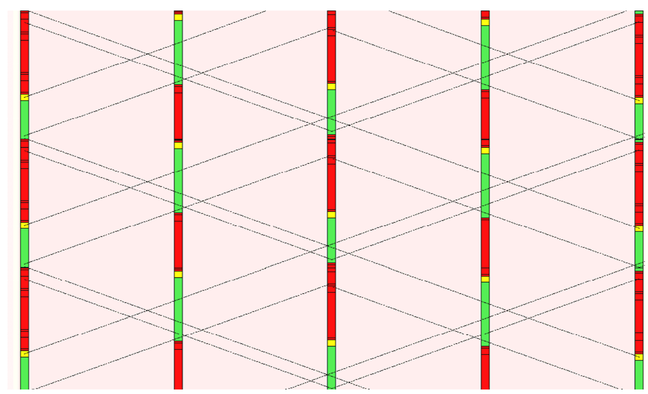
STPD tools typically produce graphic displays depicting progression characteristics. The most common display is the time–space diagram, which is well documented in the literature and understood by all practitioners. The time–space diagram reflecting the initial design is shown in Exhibit 29-9. Note that, even though the traffic volumes are balanced in both directions, the design appears to favor the westbound (right-to-left) direction. Because of the symmetry of this example, it is likely that a dual solution exists that yields the same performance but that favors the eastbound direction.

Exhibit 29-9
Time–Space Diagram for the Initial Design



The time–space diagram depicting the modified progression design is shown in Exhibit 29-10. This design shows a better balance between the eastbound and westbound directions. There is good progression into the system from both ends, but the band in both directions is halted at the center intersection. The PROS accumulation is evident in the bands that progress between some of the intersections.

Exhibit 29-10
Time–Space Diagram for the Modified Progression Design



There appears to be minimal difference between the initial and modified designs. The modified design will be chosen for further investigation because it offers a better balance between the two directions. The offset changes for this design are presented in Exhibit 29-11.

Exhibit 29-11
Offset Changes for the Modified Progression Design

Intersection	Initial Offsets	Revised Offsets
1	0	0
2	34	30
3	3	76
4	31	30
5	78	0

The time–space diagram for this operation from another STPD tool is shown in Exhibit 29-12. The timing plan is the same as the plan that was depicted in Exhibit 29-10, but the format of the display differs slightly. Both the link band and the arterial band as defined previously are shown on this display. The individual signal phases are also depicted. Both types of time–space diagrams offer a manual adjustment feature whereby the offsets may be changed by dragging the signal display back and forth on the monitor screen.

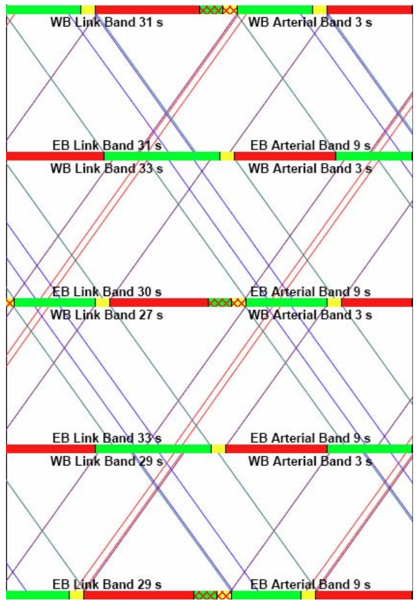


Exhibit 29-12
Alternative Time–Space Diagram Format

OTHER GRAPHIC DISPLAYS

Other graphics formats are not as ubiquitous as the time–space diagram but can provide useful insights into the operation at and between intersections.

Flow Profile Diagrams

One example is the flow profile diagram, which is simply a plot of the flow rate over one complete cycle. Flow profiles may be created to depict either the arrival or departure flows at a stop line.

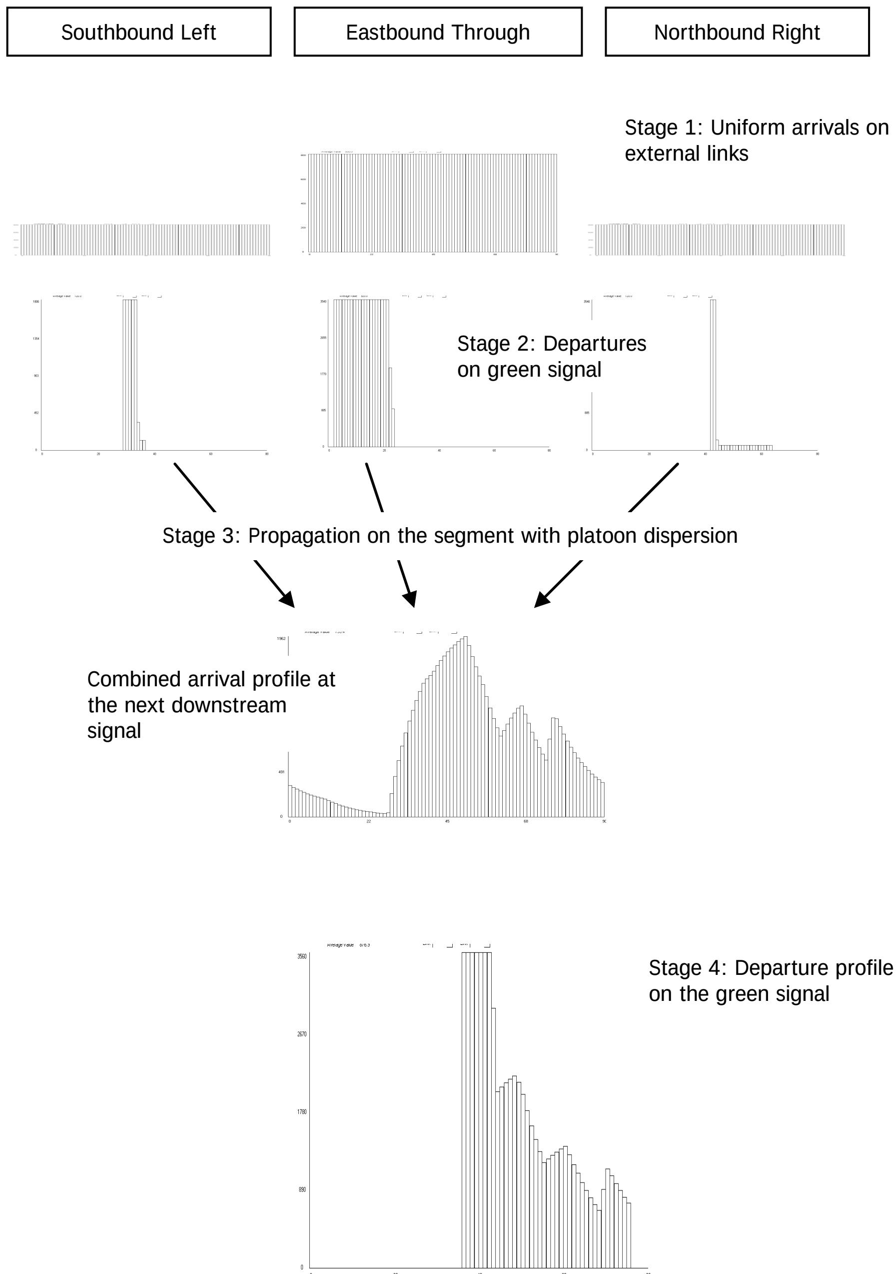
An example illustrating the use of flow profiles is presented in Exhibit 29-13. The eastbound segment between the first and second intersections is depicted in this example. The traffic inputs to this segment come from three independent movements at Intersection 1: southbound left, eastbound through, and northbound right.

Four stages of the progress of traffic into and out of this segment are depicted in the exhibit:

1. *Uniform arrivals on external links*: Each of the three movements entering the segment will arrive with a flow profile that is constant throughout the cycle because of the absence of platoon-forming phenomena on external links.
2. *Departures on the green signal*: Each movement proceeds on a different phase and therefore enters the link at a different time.
3. *Propagation on the segment with platoon dispersion*: Each of the three movements will be propagated downstream to the next signal by using a model that applies the design speed and incorporates platoon dispersion. Arrival of the platoons at the downstream end of the segment: The composite arrival profile is illustrated in the figure. The profile represents the sum of all of the movements entering the link.

4. *Departure on the green signal*: The platoons are regrouped at this point into a new flow profile because of the effect of the signal. The extent of regrouping will depend on the proportion of time that the signal is green. If a continuous green signal were displayed, the output flow profile would match the input flow profile exactly.

Exhibit 29-13
Example Illustrating the Use of Flow Profiles



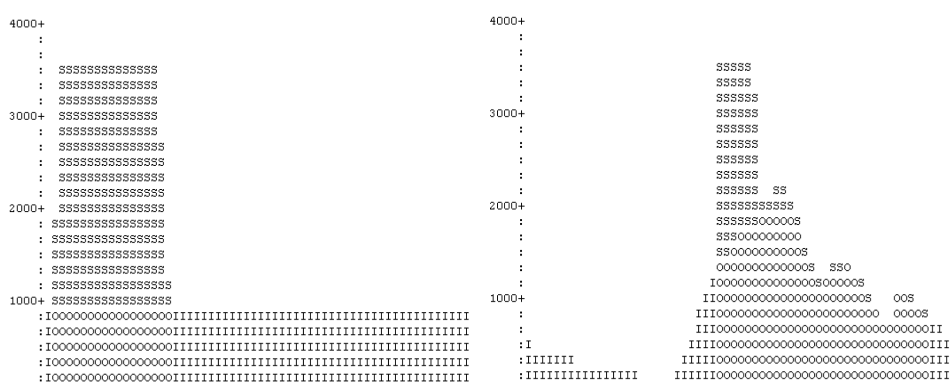
The departure profile for this movement forms one input to the next link and is therefore equivalent to Stage 2 in the list above. The vehicles entering on different phases from the cross street must be added to this movement to form the input to the next segment as the process repeats itself throughout the facility.

The preceding description of the accumulation, discharge, and propagation characteristics of flow profiles is of special interest to this discussion because the same models used by the STPD tool have been adopted by the analysis procedures given in Chapter 17, Urban Street Segments. These procedures are described by Exhibit 30-3 through Exhibit 30-5 in Chapter 30, Urban Street Segments: Supplemental. Therefore the graphical representations given in Exhibit 29-13 should provide a useful supplement to facilitate understanding of the procedures prescribed in Chapter 17.

Composite Flow Profiles

Another form of flow profile graphics is illustrated in Exhibit 29-14. This text-based display offers a composite view of the flow profiles by showing the arrival and departure graphics on the same figure represented by different characters. The uniform arrival pattern from the external link is evident at the upstream intersection, which corresponds to Stages 1 and 2 of Exhibit 29-13. The effect of the platooned arrivals is also evident at the downstream intersection, corresponding to Stages 4 and 5. More details on interpreting the composite flow profiles are given elsewhere (*3*).

Exhibit 29-14
Composite Flow Profiles for the First Eastbound Segment

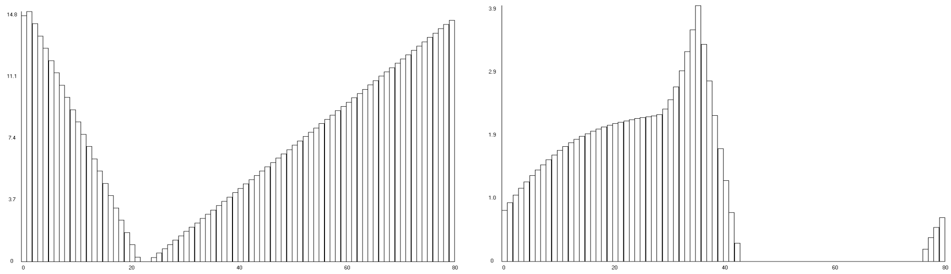


(a) Upstream Intersection (Uniform Arrivals) (b) Downstream Intersection (Platooned Arrivals)

Queue Length Graphics

The accumulation and discharge of queues can also be represented graphically in a manner that is consistent with the analysis procedures of Chapters 16 through 18. An example of graphics depicting the queue length throughout the cycle is presented in Exhibit 29-15. The upstream signal shows the familiar triangular shape that is the basis of the uniform delay equation. The downstream signal shows the effect of platooned arrivals on the length of the queue.

Exhibit 29-15
Variation of Queue Length Throughout the Signal Cycle for the First Eastbound Segment



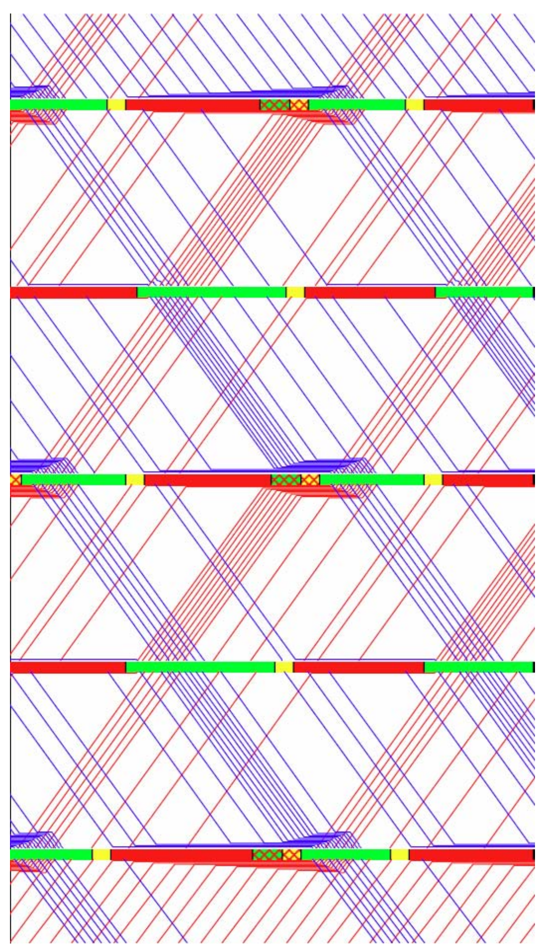
(a) Upstream Intersection Queue Length (Uniform Arrivals)

(b) Downstream Intersection (Platooned Arrivals)

Adding Flows and Queues to the Time–Space Diagram

One useful display superimposes the flow profiles and queuing characteristics on the time–space diagram to give a complete picture of the operation of the facility. An example of this display representing the improved progression design is presented in Exhibit 29-16. The flow rates are represented by the density of the lines progressing between intersections at the design speed. The queues are represented by horizontal lines upstream of each intersection. From this diagram, it is possible to visualize the effect of the design on queue accumulation and discharge and on the propagation of flows between intersections.

Exhibit 29-16
Time–Space Diagram with Flows and Queues



POTENTIAL IMPROVEMENTS FROM PHASING OPTIMIZATION

The quality of progression in this example was improved from the initial design, but the results leave room for further improvement. There are, for example, minimal arterial through bands. The current design was based on leading phases for all protected left turns. The operation might be improved by the application of lagging left-turn phases on some approaches. The procedures given in Chapter 17 are sensitive to the phase order. These procedures could be applied manually to seek a better operation. The use of STPD tools for this purpose will be demonstrated here because phasing optimization is internalized in the tools as a computational feature.

The phasing optimization process recommended changes at two of the five intersections. The phasing modifications are shown in Exhibit 29-17. Lead-lag phasing was applied at both intersections. As a result of the optimization, the arterial bandwidth increased from 6 to 16 s in both directions. The total signal delay decreased from 220 to 200 s/veh. The arterial speed increased from 22.1 to 23.0 mi/h. Thus, it is clear that the phasing optimization would improve both the progression quality and the operational performance of the route. The progression quality improvement is evident in the time–space diagram presented in Exhibit 29-18.

Exhibit 29-17
Optimized Phasing Modifications

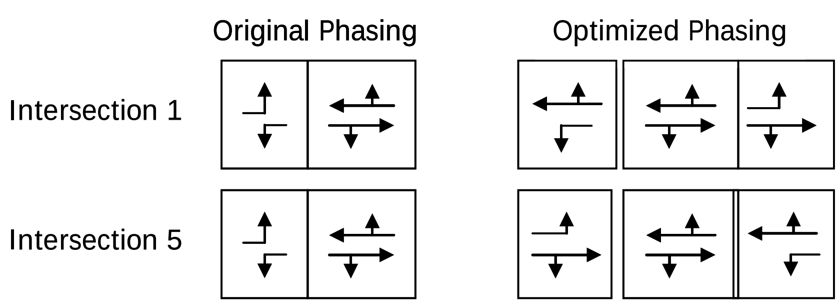
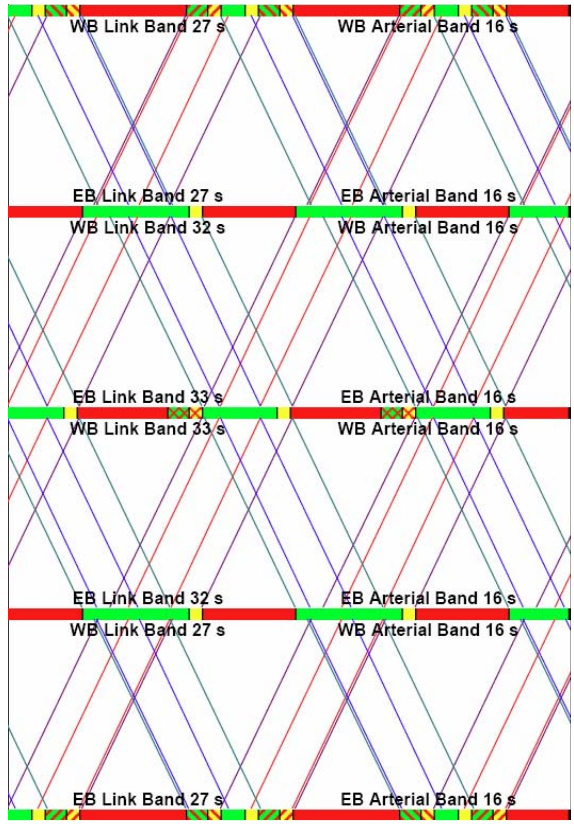


Exhibit 29-18
Time–Space Diagram for the Optimized Phasing Plan



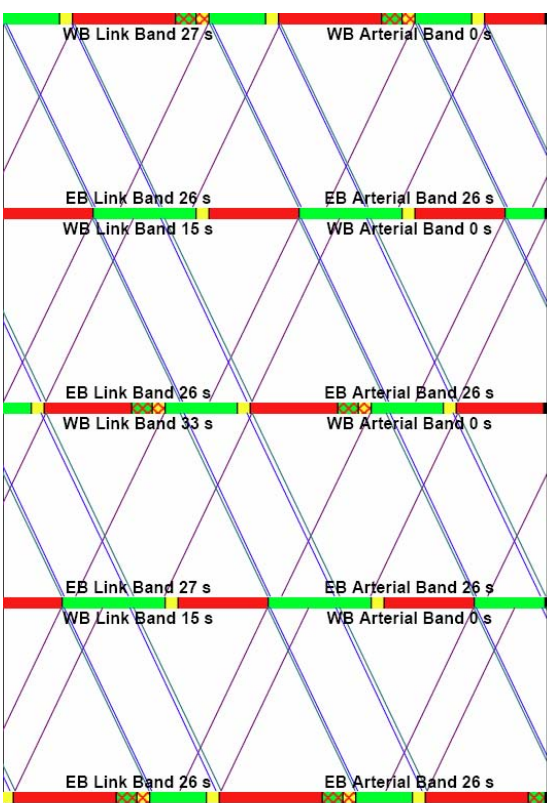
The decision to implement lead-lag phasing involves many factors including safety and local preferences. This discussion has been limited to a demonstration of how STPD tools can be used in the assessment of the operational effects of phasing optimization as one input to the decision process. The suggested modifications will not be implemented in the balance of the examples.

4. EFFECT OF MIDSEGMENT PARKING ACTIVITIES

HCM procedures recognize some midsegment activities such as cross-street entry between signals and access-point density. A procedure is provided in the methodology for estimating the delay due to vehicles turning left or right into an access-point approach. No procedures are included for estimating the delay or stops due to other causes such as pedestrian interference and parking maneuvers. Alternative tools must be used to assess these effects.

This section will demonstrate the use of a typical microscopic simulation tool (*4*) to assess the effects of midsegment parking maneuvers on the performance of an urban street facility. The signal timing plan example from the previous section of this chapter will be used for this purpose. The offsets will be modified first to create "ideal" progression in the eastbound direction at the expense of the westbound flow. The investigation will focus on the eastbound flow. The offsets and time–space diagram depicting this operation are shown in Exhibit 29-19. Offset 1 is referenced to the first phase for arterial through movements. Offset 2 is referenced to Phase 1. Their values will differ because of leading left-turn phases at some intersections. Different tools require different offset references.

Exhibit 29-19
Time–Space Diagram Showing Ideal Eastbound Progression



Signal	Offset 1	Offset 2
1	0	0
2	35	47
3	63	68
4	23	35
5	57	56

The treatment of parking maneuvers by the selected simulation tool is described in the tool's user guide (*4*). The following parameters must be supplied for each segment that contains on-street parking spaces:

- Beginning of the parking area with respect to the downstream end of the segment,

- Length of the parking area,
- Mean duration of a parking maneuver, and
- Mean frequency of parking maneuvers.

The occurrence and duration of parking maneuvers are randomized around their specified mean values. The parameters that will be used in this example are shown in Exhibit 29-20.

Exhibit 29-20
Parameters for the Parking Example

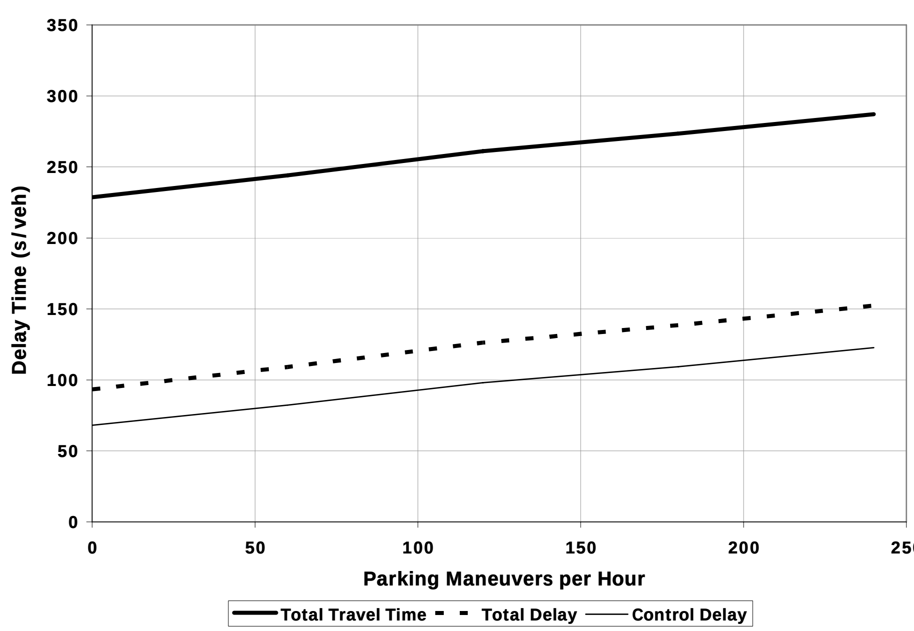
Parameter	Value
Beginning of the parking area	200 ft from the downstream intersection
Length of the parking area	1,600 ft (leaving 200 ft to the upstream intersection)
Mean duration of a parking maneuver	30 s
Mean frequency of parking maneuvers	0 veh/h (no parking maneuvers) 60 veh/h 120 veh/h 180 veh/h 240 veh/h Represents a range of approximately 15 min to 60 min average parking duration

The simulation runs covered 80 cycles of operation. Separate runs were made for each level of parking frequency. The default simulation parameters of the selected tool were used.

The effect of the parking activity on travel time and delay is presented in Exhibit 29-21, which shows the total travel time for the facility as well as the two delay components of travel time (total delay and control delay). Each of the values represents the sum of the individual segment values. The graphs demonstrate that all of the relationships were more or less linear with respect to the parking activity level.

Exhibit 29-21
Effect of Parking Activity Level on Travel Time and Delay

LIVE GRAPH
Click here to view



The effect of the parking activity on stops is presented in Exhibit 29-22. For this example, the average percentage of stops for all eastbound vehicles increased from slightly more than 40% to slightly less than 60% throughout the range of parking activity levels. Both of these exhibits indicate that the simulation tool was able to extend the capability for analysis of urban street facilities beyond the stated limitations of the methodology presented in Chapter 16.

Exhibit 29-22
Effect of Parking Activity Level on the Percentage of Stops

LIVE GRAPH
Click here to view

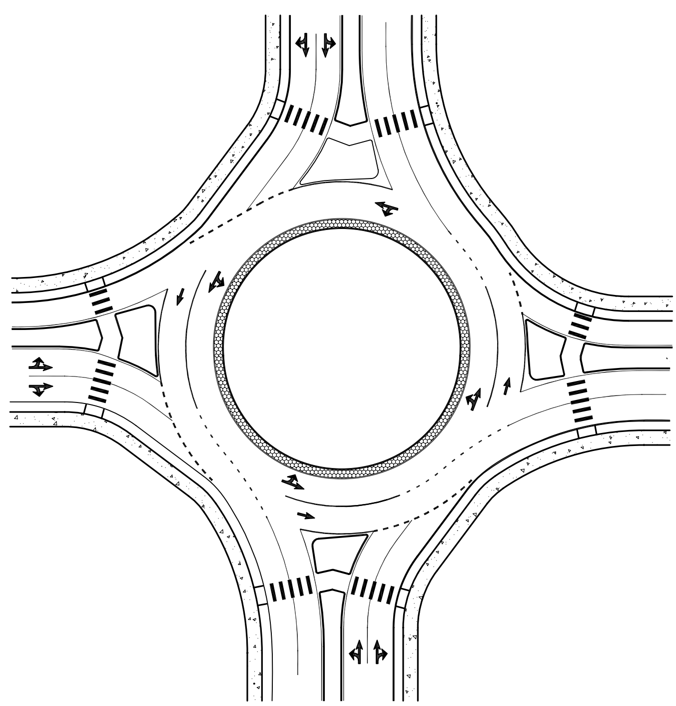
5. EFFECT OF PLATOONED ARRIVALS AT A ROUNDABOUT

Chapter 21, Roundabouts, describes a procedure for analyzing the operation of an isolated roundabout. The procedure does not deal with a roundabout operating within a signalized arterial, so no such treatment has been incorporated into the urban street segments analysis procedure. Therefore, the analysis of a roundabout as a part of a coordinated traffic control system is likely better accomplished with alternative tools. The alternative deterministic tools described earlier in this chapter do not deal explicitly with roundabouts in coordinated systems. Most simulation tools offer some roundabout modeling capability, although the level of modeling detail varies among tools.

This section describes the use of a typical simulation tool (*5*) in analyzing a roundabout within the arterial configuration of the previous example in this chapter. For this purpose, Intersection 3 at the center of the system will be converted to a roundabout with two lanes on each approach. To simplify the discussion, a basic symmetrical configuration will be used, because the discussion will be limited to the effect of platooned arrivals on the operation. The design aspects of roundabouts are covered in Chapter 21, Roundabouts, with more details provided in Chapter 33, Roundabouts: Supplemental, and elsewhere (*6*). The default traffic modeling parameters of the simulation tool will be applied.

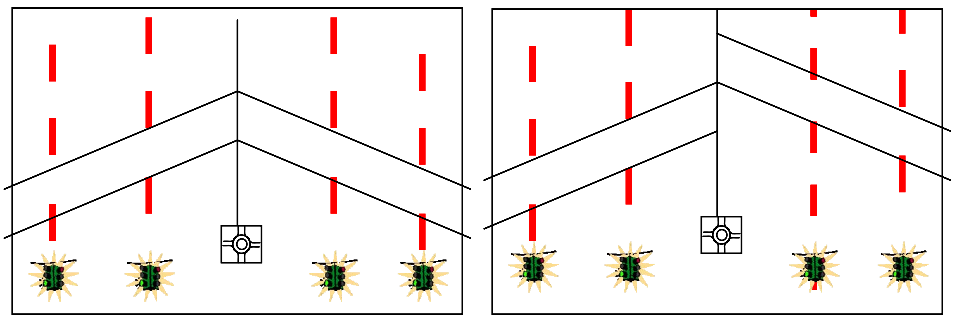
The roundabout configuration is shown schematically in Exhibit 29-23.

Exhibit 29-23
Roundabout Configuration for Intersection 3



This example will examine two STPDs that create substantially different platoon arrival characteristics on the arterial approaches to the roundabout. The time–space diagrams representing the two designs are shown in Exhibit 29-24. The first design provides simultaneous arrival of the arterial platoons from both directions. The second creates a situation in which one platoon will arrive in the first half of the cycle and the other will arrive during the second half. The two cases will be described as "simultaneous" and "alternating" platoon arrivals.

Exhibit 29-24
Time–Space Diagrams Showing Simultaneous and Alternating Platoon Arrivals at the Roundabout



(a) Simultaneous Platoon Arrivals　　(b) Alternating Platoon Arrivals

The platoon arrival characteristics can only be expected to influence the operation of a roundabout with relatively free-flowing traffic. While a two-lane roundabout could accommodate the demand volumes used in the previous examples in which the intersection was signalized, the initial simulation runs indicated enough queuing on all approaches to obscure the effect of the progression design. Since the focus of this example is on the effect of the adjacent signal timing plan, the demand volumes on the cross-street approaches to the roundabout will be reduced by 100 veh/h (approximately 17%) to provide a better demonstration of that effect.

Ten simulation runs were performed for both progression designs, and the average values of the performance measures were used to compare the two designs. The performance measures illustrated in Exhibit 29-25 include delay and stops on all approaches to the roundabout and travel times on individual link segments and on the route as a whole.

Exhibit 29-25
Performance Comparison for Simultaneous and Alternating Platoon Arrivals at a Roundabout

Movement	Alternate	Simultaneous	Difference	Percent
Delay				
Major-street approaches	15.81	14.18	1.64	10.34
Minor-street approaches	19.36	19.88	–0.52	–2.69
Stops				
Major-street approaches	0.59	0.52	0.08	12.71
Minor-street approaches	0.88	0.89	–0.01	–0.57
Average Travel Times				
Through vehicles traveling the full route	250.60	237.74	12.86	5.13
Approach links	58.06	56.30	1.76	3.03
Exit links	50.76	45.66	5.10	10.05

As a general observation, the simultaneous design performed noticeably better than did the alternating design on the major street, with a slight degradation to the cross-street performance. Travel times for vehicles traveling the full length of the facility were improved by about 5%. Travel times on the

arterial segments entering and leaving the roundabout were improved by 3% and 10%, respectively.

This example has demonstrated that the simulation tool was able to describe the effect of two signal progression schemes on the performance of a roundabout within a coordinated arterial signal system. The next example will deal with the same basic arterial layout except that the roundabout will be replaced by a two-way STOP-controlled (TWSC) intersection. The platoon arrival types can be expected to have a greater influence on the TWSC operation than the roundabout because the effect is much more direct. Major-street vehicles always have the right-of-way over minor-street vehicles. Simultaneous platoons arriving from both directions will provide more opportunity for gaps in the major-street flow. Alternating platoons will keep major-street vehicles in the intersection for a greater proportion of time, thereby restricting cross-street access.

The effect at a roundabout is much more subtle because minor-street vehicles have the right-of-way over major-street vehicles once they have entered the roundabout. With simultaneous arrivals, platoons from opposite directions assist each other by keeping the minor-street vehicles from entering and seizing control of the roadway. When there is no traffic from the opposite direction, as in the case of alternating arrivals, a major-street movement is more likely to encounter minor-street vehicles within the roundabout. This phenomenon explains the 10% improvement in performance for simultaneous arrivals in the roundabout example as indicated in Exhibit 29-25.

6. QUEUE LENGTH ANALYSIS BASED ON VEHICLE TRAJECTORIES

The HCM's segment-based chapters provide deterministic procedures for estimating the extent of queue backup on either signalized or unsignalized approaches. Most of the procedures are sensitive to some degree to platoon formation from adjacent signals. Most provide estimates of the average back of queue (BOQ) and the expected BOQ at some level of probability.

One additional queuing measure that can be derived from simulation is the proportion of time that the BOQ might be expected to extend beyond a specified point. This measure can be obtained directly from the analysis of individual vehicle trajectories by using the procedures set forth in Chapter 7, Interpreting HCM and Alternative Tool Results, and Chapter 24, Concepts: Supplemental. Those procedures will be applied in this example to examine the queuing characteristics on the minor-street approach to a TWSC intersection operating within a signalized arterial system. The criteria and procedures prescribed in Chapter 24 for identifying the onset and release from the queued state will be used.

The same urban street configuration will be used for this purpose. The center intersection that was converted to a roundabout in the previous example will now be converted to TWSC. Because of the unique characteristics of TWSC, a few changes will have to be made to the configuration. Because TWSC capacities are lower than those of signals or roundabouts, it will be necessary to reduce the minor-street demand volumes. The two-lane approaches will be preserved, but the additional left-turn bay will be eliminated. The same two platoon arrival configurations (simultaneous and alternating) will be examined to determine their effect on the minor-street queuing characteristics. The signal timing plans from the roundabout example, as illustrated in Exhibit 29-24, will also be used here. Twelve cycles covering 960 s will be simulated for each case to be examined, and the individual vehicle trajectories will be recorded.

QUEUING CHARACTERISTICS

The first part of this example will demonstrate TWSC operation with an idealized scenario to provide a starting point for more practical examples. Two intersecting streams of through movements with completely uniform characteristics will be simulated. As many of the stochastic features of the simulation model as possible will be disabled. This is a highly theoretical situation with no real practical applications in the field. Its purpose is to provide a baseline for comparison.

The formation of queues under these conditions is illustrated in Exhibit 29-26, which shows the instantaneous BOQ at all time steps in the simulation. The cross-street entry volume was 600 veh/h in each direction, representing approximately the capacity of the approach. The cyclical operation is quite evident here, with 12 discernible cycles observed. Each cycle has a similar appearance. The differences among cycles are due to embedded stochastic features that could not be disabled.

Exhibit 29-26
Queuing Results for the Theoretical Example

LIVE GRAPH
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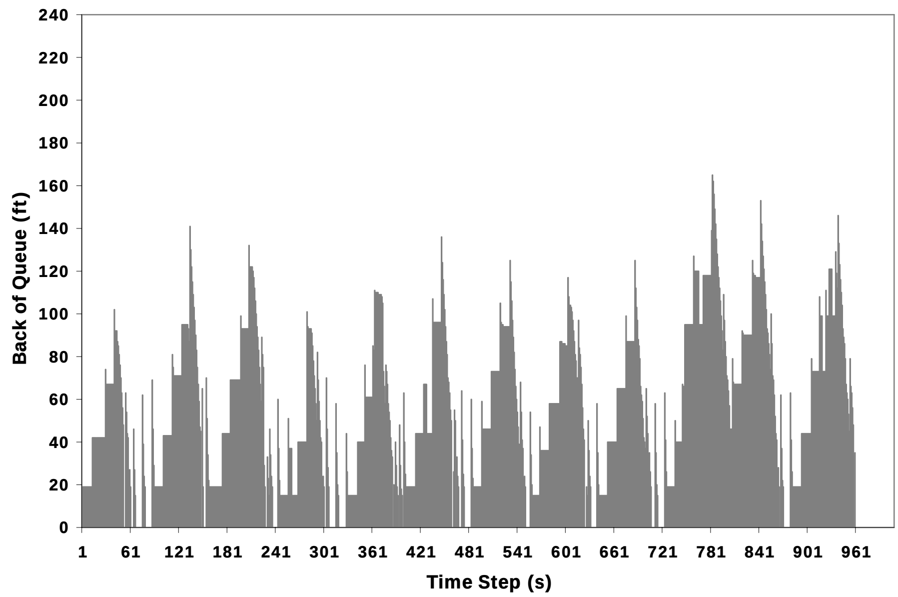

The signal timing plan with simultaneous platoon arrivals should produce the most cyclical operation that could actually be observed in the field. This configuration was simulated by loading the minor street to near capacity levels as determined experimentally. The entry volume was 350 veh/h.

The queuing results are shown in Exhibit 29-27. Some cyclical characteristics are still evident here, but they are considerably diminished from the idealized case. The loss of cyclical characteristics results from cross-street turning movements entering the segments at their upstream intersections and from the general stochastic nature of simulation modeling.

Exhibit 29-27
Queuing Results for Simultaneous Platoons

LIVE GRAPH
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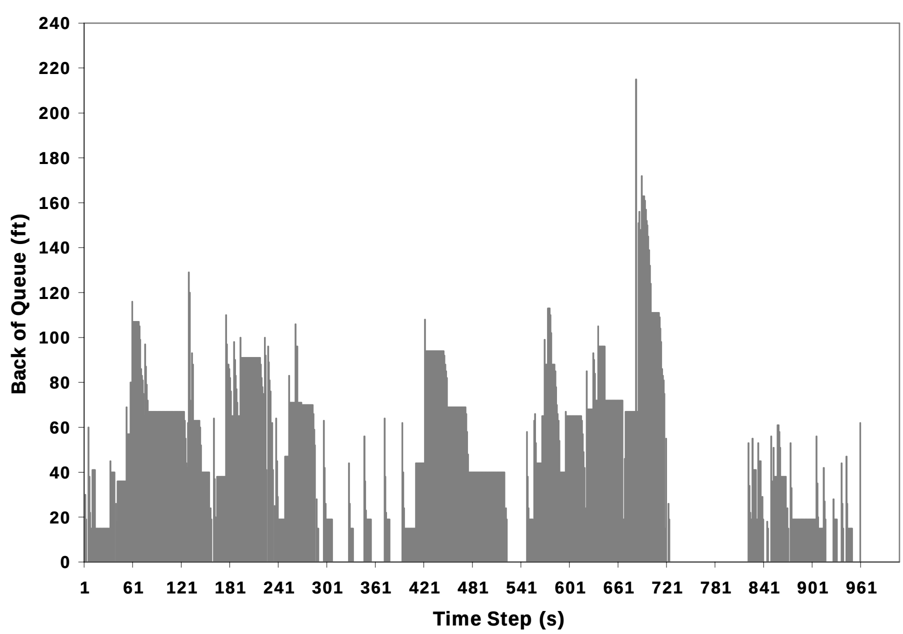

The operation was simulated next with alternating platoon arrivals. Again the demand volumes were set to the experimentally determined approach capacity, which was 270 veh/h, or about 25% lower than the capacity with simultaneous platoons. The results are presented in Exhibit 29-28. Some further loss of cyclical properties due to the spreading of entry opportunities across a greater proportion of the cycle is observed here.

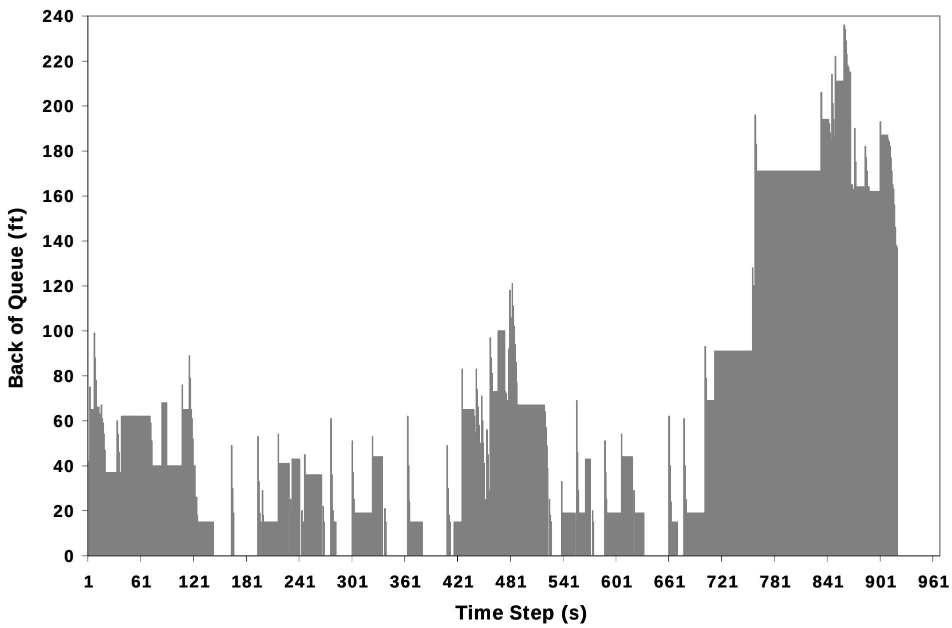

Exhibit 29-28
Queuing Results for Alternating Platoons

LIVE GRAPH
Click here to view

The least cyclical characteristics would be expected from simulation of a completely isolated operation. The 2,000-ft link lengths were retained for this case, but no adjacent intersections existed. All other parameters remained the same, including the entry volume because the entry capacity for isolated operation was found to be the same as the case with alternating platoons.

The results are presented in Exhibit 29-29. There are no cyclical characteristics here because there is no underlying cycle in the operation. Also, even with the same entry volume as the alternating platoon case, the peak BOQs are much lower. This is because the entry opportunities are distributed randomly in time instead of being concentrated at specific points in the cycle.

Exhibit 29-29
Queuing Results for Isolated TWSC Operation

LIVE GRAPH
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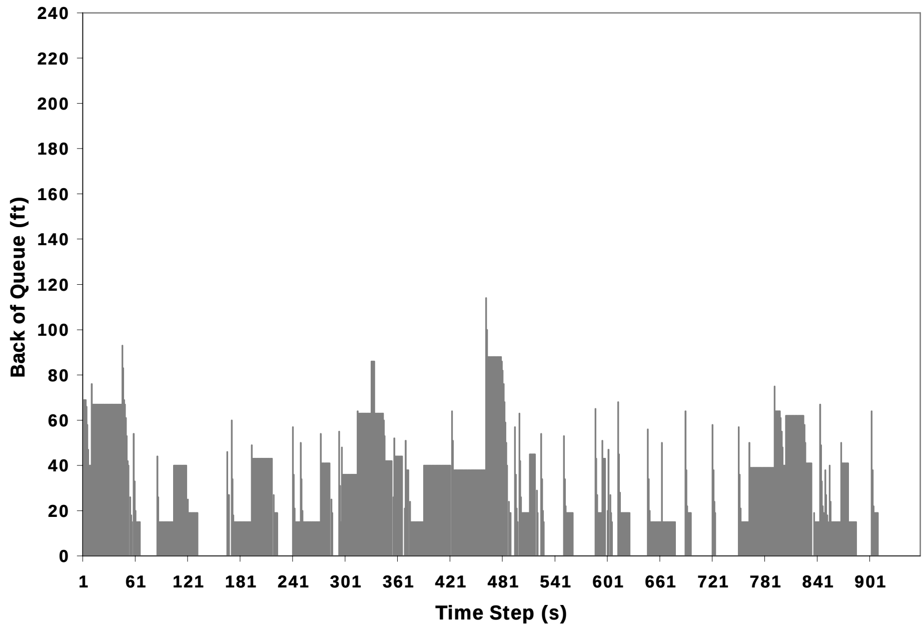

BOQ ASSESSMENT

The discussion to this point has focused on instantaneous BOQs in an effort to understand the general nature of queuing under the conditions that were examined. With knowledge of the instantaneous BOQ values available from simulation, it is possible to produce useful performance measures related to queuing from simulation. One such measure is the proportion of time that a queue would be expected to back up beyond a specified point. This concept is different from the probability of backup to that point normally associated with deterministic tools. The balance of the discussion will deal with proportion of time with queue backup (PTQB) beyond a specified point.

The three cases examined in this example were simulated with cross-street demand volumes of 80, 160, 240, 320, and 400 veh/h, and the PTQB characteristics were determined by simulation for each case. The results were plotted for a specified distance of 100 ft from the stop line as shown in Exhibit 29-30. Each case is represented by a separate line that shows the percentage of time that the queue would be expected to back up beyond 100 ft from the stop line for each cross-street entry volume level. The simultaneous platoon case showed the lowest BOQ levels, starting with no time with BOQ beyond 100 ft below 240 veh/h, and reached a value of 80% of the time at the maximum volume of 400 veh/h. Predictably, the isolated case was the most susceptible to queue backup, and the alternating platoon case fell somewhere in between.

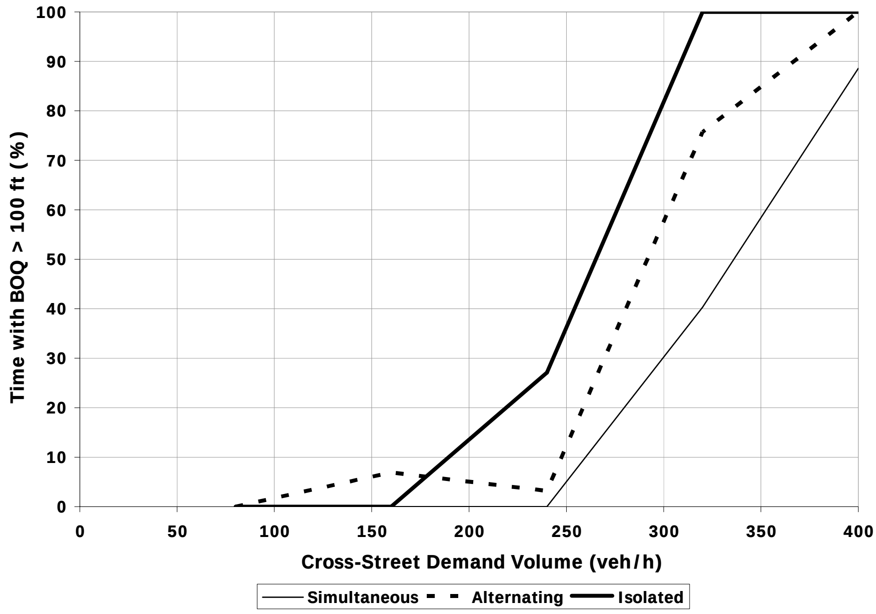

Exhibit 29-30
Effect of Cross-Street Demand Volume on Queue Backup Beyond 100 ft from the Stop Line

LIVE GRAPH
Click here to view

This example has demonstrated the use of simulation to produce potentially useful queuing measures based on the analysis of individual vehicle trajectories. It has also demonstrated how simulation can be used to assess the queuing characteristics of a minor-street approach to a TWSC intersection operating in a coordinated signal environment.

7. REFERENCES

1. Koonce, P., L. Rodegerdts, K. Lee, S. Quayle, S. Beaird, C. Braud, J. Bonneson, P. Tarnoff, and T. Urbanik. *Traffic Signal Timing Manual.* Report FHWA-HOP-08-024. Federal Highway Administration, Washington, D.C., June 2008.
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3. Wallace, C., K. Courage, M. Hadi, and A. Gan. *TRANSYT-7F User's Guide, Vol. 4 in a Series: Methodology for Optimizing Signal Timing.* University of Florida, Gainesville, March 1998.
4. *Corridor-Microscopic Simulation Program (CORSIM) Version 6.1 User's Guide.* University of Florida, Gainesville, 2008.
5. *VISSIM 5.10 User Manual.* PTV Vision, Karlsruhe, Germany, 2008.
6. Robinson, B. W., L. Rodegerdts, W. Scarbrough, W. Kittelson, R. Troutbeck, W. Brilon, L. Bondzio, K. Courage, M. Kyte, J. Mason, A. Flannery, E. Myers, J. Bunker, and G. Jacquemart. *Roundabouts: An Informational Guide.* Report FHWA-RD-00-067. Federal Highway Administration, Washington, D.C., June 2000.

CHAPTER 30
URBAN STREET SEGMENTS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. TRAFFIC DEMAND ADJUSTMENTS

This section describes adjustments made to the input vehicular demand flow rates at signalized boundary intersections such that they reasonably reflect actual operating conditions during the analysis period. These adjustments have no effect if existing vehicular flow rates are accurately quantified for the subject segment and all movements operate below their capacity. However, if the demand flow rate for one or more movements exceeds its capacity, or if there is disagreement between the count of vehicles entering and the count exiting the segment, then some movement flow rates will need to be adjusted to evaluate segment operation accurately.

This section describes three procedures that check the input flow rates and make adjustments if necessary. These procedures are

- Capacity constraint and volume balance,
- Origin–destination distribution, and
- Spillback check.

These procedures can be extended to the analysis of unsignalized boundary intersections; however, the mechanics of this extension are not described.

CAPACITY CONSTRAINT AND VOLUME BALANCE

This subsection describes the procedure for determining the turn movement flow rates at each intersection along the subject urban street segment. The analysis is separately applied to each travel direction and proceeds in the direction of travel. The procedure consists of a series of steps that are completed in sequence for the "entry" and "exit" movements associated with each segment. These movements are shown in Exhibit 30-1.

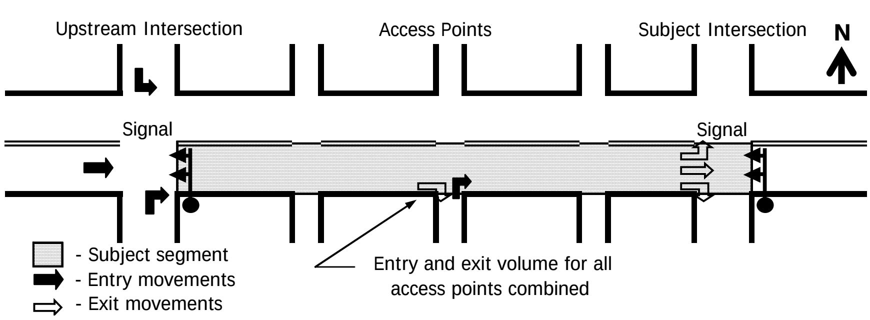


Exhibit 30-1
Entry and Exit Movements on the Typical Street Segment

As indicated in Exhibit 30-1, there are three entry movements associated with the upstream signalized intersection and three exit movements associated with the downstream signalized intersection. Entry and exit movements also exist at each access point intersection. However, these movements are aggregated into one entry and one exit movement for simplicity.

The analysis procedure is described in the following steps. Frequent reference is made to "volume" in these steps. In this application, volume is considered to be equivalent to average flow rate for the analysis period and to have units of vehicles per hour (veh/h).

Step 1: Identify Entry and Exit Volumes

Identified during this step is the volume for each entry and exit movement. The volume entering the segment from each access point intersection should be identified and added to obtain a total for the segment. Similarly, the volume exiting the segment from each access point intersection should be identified and added for the segment.

A maximum of four entry volumes are identified in this step. They include the left-turn, through, and right-turn movements at the upstream boundary intersection that enter the segment, plus the total access point entry volume. Similarly, a maximum of four exit volumes are identified. They include the left-turn, through, and right-turn movements at the downstream boundary intersection that exit the segment, plus the total access point exit volume.

Step 2: Estimate Movement Capacity

During this step, the capacity of each signalized entry movement is estimated. This estimate should be a reasonable approximation based on estimates of the saturation flow rate for the corresponding movement and the phase splits established for signal coordination.

If the right-turn movement at the upstream intersection shares a lane with its adjacent through movement, then the discharge flow rate for the turn movement can be estimated by using Equation 30-1.

Equation 30-1

$$s_{q|r} = s_{sr}\ P_R$$

where

$s_{q|r}$ = shared lane discharge flow rate for upstream right-turn traffic movement in vehicles per hour per lane (veh/h/ln),

s_{sr} = saturation flow rate in shared right-turn and through-lane group with permitted operation (veh/h/ln), and

P_R = proportion of right-turning vehicles in the shared lane (decimal).

The procedure described in Chapter 18, Signalized Intersections, is used to estimate the two variables shown in Equation 30-1. A similar equation can be constructed to estimate the shared lane discharge flow rate for an upstream left-turn movement in a shared lane.

The capacity for the right-turn movement in the shared-lane lane group is then computed by using Equation 30-2.

Equation 30-2

$$c_{q|r} = s_{q|r}\ g / C$$

where

$c_{q|r}$ = shared lane capacity for upstream right-turn traffic movement (veh/h),

$s_{q|r}$ = shared lane discharge flow rate for upstream right-turn traffic movement (veh/h/ln),

g = effective green time (s), and

C = cycle length (s).

The procedure described in Chapter 18 is used to estimate the signal timing variables shown in Equation 30-2. A similar equation can be constructed for an upstream left-turn movement in a shared lane.

Step 3: Compute Volume-to-Capacity Ratio

During this step, the volume-to-capacity ratio is computed for each signalized entry movement. This ratio is computed by dividing the arrival volume from Step 1 by the capacity estimated in Step 2. Any movements with a volume-to-capacity ratio in excess of 1.0 will meter the volume arriving to the downstream intersection.

Step 4: Compute Discharge Volume

The discharge volume from each of the three signalized entry movements is equal to the smaller of its entry volume or its associated movement capacity. The total discharge volume for the combined access point approach is assumed to be equal to the total access point entry volume. As a last calculation, the four discharge volumes are added to obtain the total discharge volume.

Step 5: Compute Adjusted Exit Volume

The total discharge volume from Step 4 should be compared with the total exit volume. The total exit volume represents the sum of the four exit volumes identified in Step 1. If the two totals do not agree in magnitude, the four exit volumes must be adjusted such that their sum equals the total discharge volume. The adjusted exit volume for a movement equals its exit volume multiplied by the “volume ratio.” The volume ratio equals the total discharge volume divided by the total exit volume.

Step 6: Repeat Steps 1 Through 5 for Each Segment

The preceding steps should be completed for each segment in the facility in the subject direction of travel. The procedure should then be repeated for the opposing direction of travel.

ORIGIN–DESTINATION DISTRIBUTION

The volume of traffic that arrives at a downstream intersection for a given downstream movement represents the combined volume from each upstream point of entry weighted by its percentage contribution to the downstream movement. The distribution of these contribution percentages between each upstream and downstream pair is represented as an origin–destination distribution matrix.

Ideally, an origin–destination survey would be conducted for an existing segment, or the origin–destination data would be available from traffic forecasts by planning models. One matrix would be available for each direction of travel on the segment. In the absence of such information, origin–destination volumes can be estimated from the entry and exit volumes for a segment, where the exit volumes equal the adjusted arrival volumes from the procedure described in the previous subsection, Capacity Constraint and Volume Balance.

Each of the four entry movements to the segment shown in Exhibit 30-1 is considered an origin. Each of the four exit movements is a destination. The problem then becomes one of estimating the origin–destination table given the entering and exiting volumes.

This procedure is derived from research (*1*). It is based on the principle that total entry volume is equal to total exit volume. It uses seed proportions to represent the best estimate of the volume distribution. These proportions are refined through implementation of the procedure. It is derived to estimate the most probable origin–destination volumes by minimizing the deviation from the seed percentages while ensuring the equivalence of entry and exit volumes.

The use of seed percentages allows the procedure to adapt the origin–destination volume estimates to factors or geometric situations that induce greater preference for some entry–exit combinations than is suggested by simple volume proportion (e.g., a downstream freeway on-ramp). The default seed proportions are listed in Exhibit 30-2.

Exhibit 30-2
Default Seed Proportions for Origin–Destination Matrix

Seed Proportion by Origin Movement				Destination Movement
Left	**Through**	**Right**	**Access Point**	
0.02	0.10	0.05	0.02	Left
0.91	0.78	0.92	0.97	Through
0.05	0.10	0.02	0.01	Right
0.02	0.02	0.01	0.00	Access point
1.00	1.00	1.00	1.00	

Step 1: Set Adjusted Origin Volume

Equation 30-3

$$O_{a,i} = O_i$$

where

$O_{a,i}$ = adjusted volume for origin i (i = 1, 2, 3, 4) (veh/h), and

O_i = volume for origin i (i = 1, 2, 3, 4) (veh/h).

The letter i denotes the four movements entering the segment. This volume is computed for each of the four origins.

Step 2: Compute Adjusted Destination Volume

Equation 30-4

$$D_{a,j} = \sum_{i=1}^{4} O_{a,i}\, p_{i,j}$$

where

$D_{a,j}$ = adjusted volume for destination j (j = 1, 2, 3, 4) (veh/h),

$O_{a,i}$ = adjusted volume for origin i (i = 1, 2, 3, 4) (veh/h), and

$p_{i,j}$ = seed proportion of volume from origin i to destination j (decimal).

The letter j denotes the four movements exiting the segment. This volume is computed for each of the four destinations.

Step 3: Compute Destination Adjustment Factor

$$b_{d,j} = \frac{D_j}{D_{a,j}}$$

Equation 30-5

where

$b_{d,j}$ = destination adjustment factor j (j = 1, 2, 3, 4),

D_j = volume for destination j (j = 1, 2, 3, 4) (veh/h), and

$D_{a,j}$ = adjusted volume for destination j (j = 1, 2, 3, 4) (veh/h).

This factor is computed for each of the four destinations.

Step 4: Compute Origin Adjustment Factor

$$b_{o,i} = \sum_{j=1}^{4} b_{d,j}\ p_{i,j}$$

Equation 30-6

where $b_{o,i}$ is the origin adjustment factor i (i = 1, 2, 3, 4), and other variables are as previously defined. This factor is computed for each of the four origins.

Step 5: Compute Adjusted Origin Volume

$$O_{a,i} = \frac{O_i}{b_{o,i}}$$

Equation 30-7

where $O_{a,i}$ = adjusted volume for origin i (i = 1, 2, 3, 4) (veh/h), and other variables are as previously defined. This volume is computed for each of the four origins. It replaces the value previously determined for this variable.

For each origin, compute the absolute difference between the adjusted origin volume from Equation 30-7 and the previous estimate of the adjusted origin volume. If the sum of these four differences is less than 0.01, then proceed to Step 6; otherwise, set the adjusted origin volume for each origin equal to the value from Equation 30-7, go to Step 2, and repeat the calculation sequence.

Step 6: Compute Origin–Destination Volume

$$v_{i,j} = O_{a,i}\ b_{d,j}\ p_{i,j}$$

Equation 30-8

where $v_{i,j}$ is the volume entering from origin i and exiting at destination j (veh/h), and other variables are as previously defined. This volume is computed for all 16 origin–destination pairs.

SPILLBACK CHECK

This subsection describes the procedure for determining whether queue spillback occurs on one or more segments of an urban street facility. The analysis is separately applied to each travel direction and proceeds in the direction of travel. The procedure consists of a series of steps that are completed in sequence for the signalized exit movements associated with each segment. These movements are shown in Exhibit 30-1. Spillback due to the movements associated with the access points is not specifically addressed.

Step 1: Identify Initial Queue

During this step, the initial queue for each signalized exit movement is identified. This value represents the queue present at the start of the analysis period (the total of all vehicles in all lanes serving the movement). The initial queue estimate would likely be available for the evaluation of an existing condition for which field observations indicate the presence of a queue at the start of the analysis period. For planning or preliminary design applications, it can be assumed to equal 0.0 vehicles.

Step 2: Identify Queue Storage Length

The length of queue storage for each exit movement is identified during this step. For turn movements served from a turn bay, this length equals the length of the turn bay. For through movements, this length equals the segment length less the width of the upstream intersection. For turn movements served from a lane equal in length to that of the segment, the queue storage length equals the segment length less the width of the upstream intersection.

Step 3: Compute Maximum Queue Storage

The maximum queue storage for the exiting through movement is computed by using Equation 30-9.

Equation 30-9

$$N_{qx,thru} = \frac{(N_{th} - P_L - P_R)\, L_{a,thru}}{L_h}$$

with

Equation 30-10

$$L_h = L_{pc}(1 - 0.01\, P_{HV}) + 0.01\, L_{HV}\, P_{HV}$$

where

$N_{qx,thru}$ = maximum queue storage for the through movement (veh),

N_{th} = number of through lanes (shared or exclusive) (ln),

P_L = proportion of left-turning vehicles in the shared lane (decimal),

P_R = proportion of right-turning vehicles in the shared lane (decimal),

$L_{a,thru}$ = available queue storage distance for the through movement (ft/ln),

L_h = average vehicle spacing in stationary queue (ft/veh),

L_{pc} = stored passenger-car lane length = 25 (ft),

L_{HV} = stored heavy-vehicle lane length = 45 (ft), and

P_{HV} = percent heavy vehicles in the corresponding movement group (%).

The procedure described in Chapter 18, Signalized Intersections, is used to estimate P_L and P_R. If there are no shared lanes, then P_L = 0.0 and P_R = 0.0.

The maximum queue storage for a turn movement is computed by using Equation 30-11.

Equation 30-11

$$N_{qx,turn} = \frac{N_{turn}\, L_{a,turn} + P_{turn}\, L_{a,thru}}{L_h}$$

where

$N_{qx,turn}$ = maximum queue storage for a turn movement (veh),

N_{turn} = number of lanes in the turn bay (ln),

$L_{a,turn}$ = available queue storage distance for the turn movement (ft/ln),

P_{turn} = proportion of turning vehicles in the shared lane = P_L or P_R (decimal), and

L_h = average vehicle spacing in stationary queue (ft/veh).

This equation is applicable to turn movements in exclusive lanes (i.e., P_{turn} = 0.0) and to turn movements that share a through lane.

Step 4: Compute Available Storage Length

The available storage length is computed for each signalized exit movement by using Equation 30-12.

Equation 30-12

$$N_{qa} = N_{qx} - Q_b \quad \geq 0.0$$

where

N_{qa} = available queue storage (veh),

N_{qx} = maximum queue storage for the movement (veh), and

Q_b = initial queue at the start of the analysis period (veh).

The analysis thus far has treated the three signalized exit movements as if they were independent. At this point, the analysis must be extended to include the combined through and left-turn movement when the left-turn movement has a bay (i.e., it does not have a lane that extends the length of the segment). The analysis must also be extended to include the combined through and right-turn movement when the right-turn movement has a bay (but not a full-length lane).

The analysis of these newly formed "combined movements" is separated into two parts. The first part represents the analysis of just the bay. This analysis represents a continuation of the exit movement analysis using the subsequent steps of this procedure. The second part represents the analysis of the length of the segment shared by the turn movement and the adjacent through movement. The following rules are used to evaluate the combined movements for the shared segment length:

1. The volume for each combined movement equals the sum of the adjusted arrival volumes for the two contributing movements. These volumes are obtained from the procedure described in the previous subsection, Origin–Destination Distribution.
2. The initial queue for each combined movement is computed by using Equation 30-13.

Equation 30-13

$$Q_{b,comb} = \max\left(0.0, Q_{b,turn} - \frac{L_{a,turn}\ N_{turn}}{L_h}, Q_{b,thru} - \frac{L_{a,turn}\ N_{th}}{L_h}\right)$$

where $Q_{b,comb}$ equals the initial queue for the combined movement (veh). The other variables were defined previously and are evaluated for the movement indicated by the variable subscript.

3. The queue storage length for a combined movement $L_{a,comb}$ equals the queue storage length for the through movement less the queue storage length of the turn movement (i.e., $L_{a,comb} = L_{a,thru} - L_{a,turn}$).
4. The number of lanes available to the combined movement N_{comb} equals the number of lanes available to the through movement.
5. The maximum queue storage for the combined movement is computed by applying the guidance provided in Step 3 using the variables from Rules 3 and 4.
6. The available storage length for the combined movement $N_{qa,comb}$ is computed by using the guidance provided previously in this step for the exit movements.

Step 5: Compute Capacity

The capacity for both the exit movements and the combined movements is established in this step. The capacity for each exit movement was computed in Step 2 in the subsection titled Capacity Constraint and Volume Balance. The capacity of the combined movements is computed by using Equation 30-14.

Equation 30-14

$$c = \frac{v_{a,1}}{X_1} + \frac{c_{thru}\,(N_{th} - 1)}{N_{th}}$$

with

Equation 30-15

$$v_{a,1} = \max\left(v_{a,turn}, \frac{v_{a,turn} + v_{a,thru}}{N_{th}} \right)$$

Equation 30-16

$$X_1 = \frac{v_{a,turn}}{c_{turn}} + \frac{v_{a,1} - v_{a,turn}}{c_{thru} / N_{th}}$$

where

c = capacity of the combined movements (veh/h),

$v_{a,1}$ = adjusted arrival volume in the shared lane (veh/h),

X_1 = volume-to-capacity ratio in the shared lane,

c_{thru} = capacity for the exiting through movement (veh/h),

c_{turn} = capacity for the exiting turn movement (veh/h),

$v_{a,turn}$ = adjusted arrival volume for the subject turn movement (veh/h),

$v_{a,thru}$ = adjusted arrival volume for the subject through movement (veh/h), and

N_{th} = number of through lanes (shared or exclusive) (ln).

The two adjusted arrival volumes $v_{a,turn}$ and $v_{a,thru}$ are obtained from the procedure described in the subsection titled Origin–Destination Distribution.

Step 6: Compute Queue Growth Rate

During this step, the queue growth rate is computed for each signalized exit movement for which the storage extends the length of the segment. Typically, the through movement satisfies this requirement. A turn movement may also satisfy this requirement if it is served by an exclusive lane that extends the length of the segment. The queue growth rate is computed as the difference between the adjusted arrival volume v_a and the capacity c for the subject exit movement. Equation 30-17 is used to compute this rate.

Equation 30-17

$$r_{qg} = v_a - c \quad \geq 0.0$$

where r_{qg} is the queue growth rate (veh/h) and other variables are as defined previously.

The queue growth rate is also computed for the combined movements formulated in Step 4. The adjusted volume used in Equation 30-17 represents the sum of the through and turn movement volumes in the combined group. The capacity for the group was computed in Step 5.

Step 7: Compute Time Until Spillback

During this step, the time until spillback is computed for each signalized exit movement for which the storage extends the length of the segment. This time is computed by using Equation 30-18 for any movement with a nonzero queue growth rate.

Equation 30-18

$$T_c = \frac{N_{qa}}{r_{qg}}$$

where T_c is the time until spillback (h) and other variables are as defined previously.

For turn movements served by a bay, the computed spillback time represents the time required for the bay to overflow. It does not represent the time that the turn-related queue reaches the upstream intersection.

Equation 30-18 is also used to compute the spillback time for the combined movements formulated in Step 4. However, this spillback time represents the additional time required for the queue to grow along the length of segment shared by the turn movement and the adjacent through movement. This time must be added to the time required for the corresponding turn movement to overflow its bay to obtain the actual spillback time for the combined movement.

Step 8: Repeat Steps 1 Through 7 for Each Segment

The preceding steps should be completed for each segment in the facility in the subject direction of travel. The procedure should then be repeated for the opposing direction of travel.

Step 9: Determine Controlling Spillback Time

During this step, the shortest time until spillback for each of the exit movements (or movement groups) for each segment and direction of travel is identified. If the segment supports two travel directions, then two values are

identified (one value for each direction). The shorter value of the two represents the controlling spillback time for the segment. If a movement (or movement group) does not spill back, it is not considered in this process for determining the controlling spillback time.

Next, the controlling segment times are compared for all segments that make up the facility. The shortest time found represents the controlling spillback time for the facility.

If the controlling spillback time exceeds the analysis period, then the results from the automobile methodology are considered to reflect the operation of the facility accurately. If spillback occurs before the end of the desired analysis period, then the analyst should consider either (*a*) reducing the analysis period such that it ends before spillback occurs or (*b*) using an alternative analysis tool that is able to model the effect of spillback conditions.

2. SIGNALIZED SEGMENT ANALYSIS

This section describes the process for analyzing vehicular traffic flow on a segment bounded by signalized intersections. Initially, this process computes the flow profile of discharging vehicles at the upstream intersection as influenced by the signal timing and phase sequence. It uses this profile to compute the arrival flow profile at a downstream junction. This arrival flow profile is then compared with the downstream signal timing and phase sequence to compute the proportion of vehicles arriving during green. The arrival flow profile is also used to compute the proportion of time that a platoon blocks one or more traffic movements at a downstream access point intersection. These two platoon descriptors are used in subsequent procedures to compute delay and other performance measures.

This section describes four procedures that are used to define the arrival flow profile and compute the related platoon descriptors. These procedures are

- Discharge flow profile,
- Running time,
- Projected arrival flow profile, and
- Proportion of time blocked.

Each procedure is described in the following subsections.

DISCHARGE FLOW PROFILE

A flow profile is a macroscopic representation of steady traffic flow conditions for the average signal cycle during the specified analysis period. The cycle is represented as a series of 1-s time intervals (hereafter referred to as "time steps"). The start time of the cycle is 0.0 s, relative to the system reference time. The time steps are numbered from 1 to C', where C' is the cycle length in units of time steps. The flow rate for step i represents an average of the flows that occur during the time period corresponding to step i for all cycles in the analysis period. This approach is conceptually the same as that used in the TRANSYT-7F model (*2*).

A discharge flow profile is computed for each of the upstream left-turn, through, and right-turn movements. Each profile is defined by the time that the signal is effectively green and by the time that the queue service time ends. During the queue service time, the discharge flow rate is equal to the saturation flow rate. After the queue service time is reached, the discharge rate is set equal to the "adjusted discharge volume." The adjusted discharge volume is equal to the discharge volume computed by using the procedures described in Section 1, but it is adjusted to reflect the "proportion of arrivals during green." This latter adjustment adapts the discharge flow pattern to reflect platoon arrivals on the upstream segment.

The discharge flow profile is dependent on movement saturation flow rate, queue service time, phase duration, and proportion of arrivals during green for the discharging movements. The movement saturation flow rate is computed by

using a procedure described in Section 1. Procedures for calculating the remaining variables are described in subsequent subsections. This relationship introduces a circularity in the computations that requires an iterative sequence of calculations to converge on the steady-state solution.

RUNNING TIME

The running time procedure describes the calculation of running time between the upstream intersection and a downstream intersection. This procedure is described as Step 2 of the automobile methodology in Chapter 17, Urban Street Segments.

One component of running time is the delay due to various midsegment sources. One notable source of delay is left or right turns from the segment at an access point intersection. This delay is computed by using the procedure described in Section 3. Other sources of delay include on-street parking maneuvers and pedestrian crosswalks. Delay from these sources represents an input variable to the methodology.

PROJECTED ARRIVAL FLOW PROFILE

This subsection describes the procedure for predicting the arrival flow profile at a downstream intersection (i.e., access point or boundary intersection). This flow profile is based on the discharge flow profile and running time computed previously. The discharge flow profile is used with a platoon dispersion model to compute the arrival flow profile. The platoon dispersion model is summarized in the next part of this subsection. The procedure for using this model to estimate the arrival flow profile is described in the second part.

Platoon Dispersion Model

The platoon dispersion model was originally developed for use in the TRANSYT model (*3*). Input to the model is the discharge flow profile for a specified traffic movement. Output statistics from the model include (*a*) the arrival time of the leading vehicles in the platoon to a specified downstream intersection and (*b*) the flow rate during each subsequent time step.

In general, the arrival flow profile has a lower peak flow rate than the discharge flow profile owing to the dispersion of the platoon as it travels down the street. Also, for similar reasons, the arrival flow profile is spread out over a longer period of time than the discharge flow profile. The rate of dispersion increases with increasing segment running time, which may be caused by access point activity, on-street parking maneuvers, and other midsegment delay sources.

The platoon dispersion model is described by Equation 30-19.

Equation 30-19

$$q'_{a|u,j} = F\, q'_{u,i} + (1 - F)\, q'_{a|u,j-1}$$

with

Equation 30-20

$$j = i + t'$$

where

$q'_{a|u,j}$ = arrival flow rate in time step j at a downstream intersection from upstream source u (veh/step),

$q'_{u,i}$ = departure flow rate in time step i at upstream source u (veh/step),

F = smoothing factor,

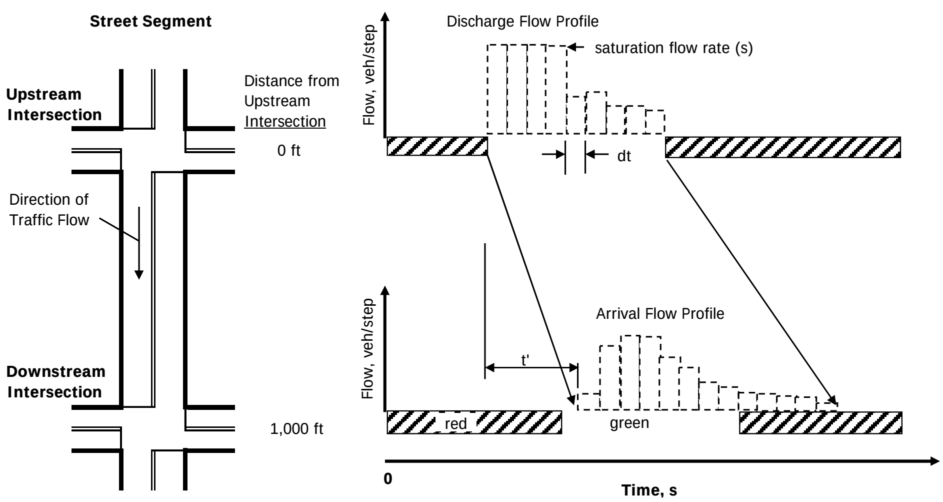
j = time step associated with platoon arrival time t', and

t' = platoon arrival time (steps).

The upstream flow source u can be either the left-turn, through, or right-turn movement at the upstream boundary intersection. It can also be the collective set of left-turn or right-turn movements at access point intersections between the upstream boundary intersection and the subject intersection.

Exhibit 30-3 illustrates an arrival flow profile obtained from Equation 30-19. In this figure, the discharge flow profile is input to the model as variable $q'_{u,i}$. The dashed rectangles that form the discharge flow profile indicate the flow rate during each of nine time steps (i = 1, 2, 3, . . . , 9) that are each d_t seconds in duration. The vehicles that depart in the first time step (i = 1) arrive at the downstream intersection after traveling an amount of time equal to t' steps. The arrival flow at any time step j (= i + t') is computed by using Equation 30-19.

Exhibit 30-3
Platoon Dispersion Model



Research (*4*) indicates that Equation 30-21 describes the relationship between the smoothing factor and running time.

Equation 30-21

$$F = \frac{1}{1 + 0.138\, t'_R + 0.315 / d_t}$$

where

t'_R = segment running time = t_R/d_t (steps),

t_R = segment running time (s), and

d_t = time step duration (s/step).

The recommended time step duration for this procedure is 1.0 s/step. Shorter values can be rationalized to provide a more accurate representation of the profile, but they also increase the time required for the computations. Experience

indicates that 1.0 s/step provides a good balance between accuracy and computation time.

Equation 30-22 is used to compute platoon arrival time to the subject downstream intersection.

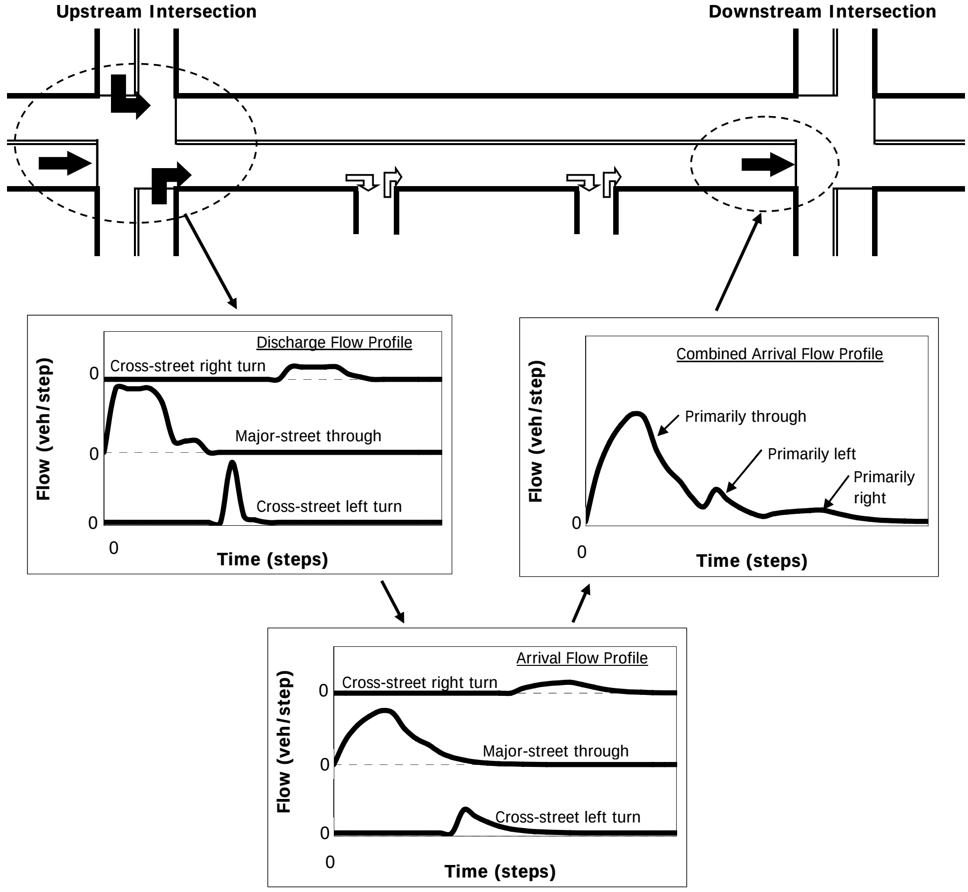
Equation 30-22

$$t' = t'_R - \frac{1}{F} + 1.25$$

Arrival Flow Profile

This part describes the procedure for computing the arrival flow profile. Typically, there are three upstream traffic movements that depart at different times during the signal cycle; they are the minor-street right turn, major-street through, and minor-street left turn. Traffic may also enter the segment at various midblock access points. Exhibit 30-4 illustrates how these movements join to form the arrival flow profile for the subject downstream intersection.

Exhibit 30-4
Arrival Flow Profile Estimation Procedure



In application, the discharge flow profile for each of the departing movements is obtained from the discharge flow profile procedure described previously. These profiles are shown in the first of the three x-y plots in Exhibit 30-4. The platoon dispersion model is then used to estimate the arrival flows for each movement at a downstream intersection. These arrival flow profiles are shown in the second x-y plot in the exhibit. Although not shown, arrivals from

midsegment access points are assumed to have a uniform arrival flow profile (i.e., a constant flow rate for all time steps).

Finally, the origin–destination distribution procedure is used to distribute each arrival flow profile to each of the downstream exit movements. The four arrival flow profiles associated with the subject exit movement are added together to produce the combined arrival flow profile. This profile is shown in the third *x-y* plot. The upstream movement contributions to this profile are indicated by arrows.

Comparison of the profiles in the first and second *x-y* plots of Exhibit 30-4 illustrates the platoon dispersion process. In the first *x-y* plot, the major-street through movement has formed a dense platoon as it departs the upstream intersection. However, by the time this platoon reaches the downstream intersection it has spread out and has a lower peak flow rate. In general, the amount of platoon dispersion increases with increasing segment length. For very long segments, the platoon structure degrades and arrivals become uniform throughout the cycle.

Platoon structure can also degrade as a result of significant access point activity along the segment. Streets with frequent active access point intersections tend to have more vehicles leave the platoon (i.e., turn from the segment at an access point) and enter the segment after the platoon passes (i.e., turn in to the segment at an access point). Both activities result in significant platoon decay.

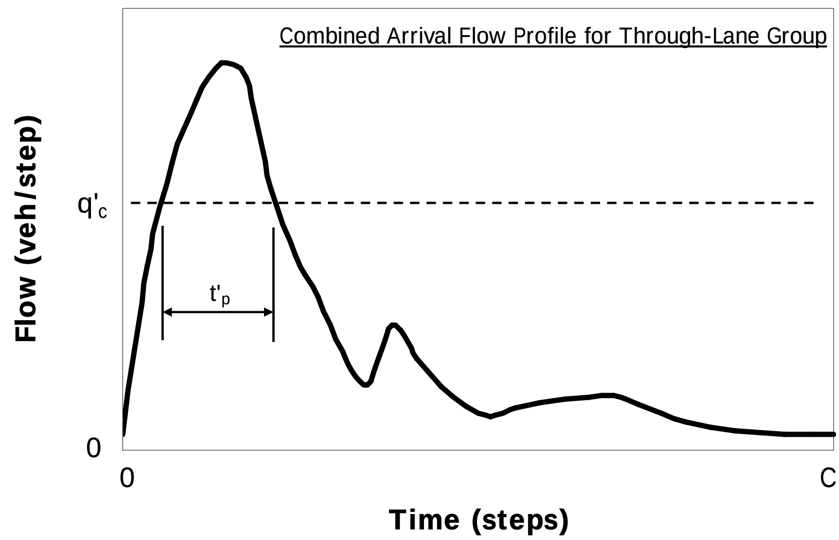
The effect of platoon decay is modeled by using the origin–destination matrix, in which the combined access point activity is represented as one volume assigned to midsegment origins and destinations. When the access point volume is large, it corresponds to a smaller volume that enters at the upstream boundary intersection as a defined platoon. This results in a larger portion of the combined arrival flow profile defined by uniform (rather than platoon) arrivals. When a street has busy access points, platoon decay tends to be a more dominant cause of platoon degradation than platoon dispersion.

PROPORTION OF TIME BLOCKED

The combined arrival flow profile can be used to estimate the time that a platoon passes through a downstream access point intersection. During this time period, the platoon can be sufficiently dense as to preclude a minor movement driver from finding an acceptable gap.

The use of the arrival flow profile to estimate the blocked period duration is shown in Exhibit 30-5. The profile shown represents the combined arrival flow profile for the through-lane group at a downstream access point intersection. The dashed line represents the critical platoon flow rate. Flow rates in excess of this threshold are rationalized to be associated with platoon headways that are too short to be entered (or crossed) by minor movements. The critical platoon flow rate q_c is equal to the inverse of the critical headway t_c associated with the minor movement (i.e., $q_c = 3{,}600/t_c$). The appropriate critical headway values for various movements are identified in Chapter 19, Two-Way STOP-Controlled Intersections.

Exhibit 30-5
Estimation of Blocked Period Duration



In the situation of a driver desiring to complete a left turn from the major street across the traffic stream represented by Exhibit 30-5, the proportion of time blocked is computed by using Equation 30-23. For this maneuver, the blocked period duration is based on the flow profile of the opposing through-lane group.

Equation 30-23

$$p_b = \frac{t'_p \, d_t}{C}$$

where

p_b = proportion of time blocked (decimal),

t'_p = blocked period duration (steps),

d_t = time step duration (s/step), and

C = cycle length (s).

Equation 30-23 is also used for the minor-street right-turn movement. However, in this situation, the blocked period duration is computed for the through-lane group approaching from the left. For the minor-street left-turn and through movements, the arrival flow profiles from both directions are evaluated. In this instance, the blocked period duration represents the time when a platoon from either direction is present in the intersection.

3. DELAY DUE TO TURNS

This section describes a process for estimating the delay to through vehicles that follow vehicles turning from the major street into an unsignalized access point intersection. This delay can be incurred at any access point intersection along the street. For right-turn vehicles, the delay results when the following vehicles' speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can also occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

The delay estimation process consists of the following two procedures:

- Delay due to left turns, and
- Delay due to right turns.

Each procedure is described in the following subsections. These procedures are based on the assumption that the segment traffic flows are random. While this assumption may not be strictly correct for urban streets, it is conservative in that it will yield slightly larger estimates of delay. Moreover, expansion of the models to accommodate platooned flows would not likely be cost-effective given the small amount of delay caused by turning vehicles.

DELAY DUE TO LEFT TURNS

Through vehicles on the major-street approach to an unsignalized intersection can incur delay when the left-turn queue exceeds the available storage and blocks the adjacent through lane (in this context, the undivided cross section is considered a major-street approach having no left-turn storage). The through vehicles that follow are delayed when they stop behind the queue of turning vehicles. This delay ends when the left-turn vehicle departs or the through vehicle merges into the adjacent through lane. By merging into the adjacent lane, drivers reduce their delay relative to the delay they would have incurred had they waited for the left-turn queue to clear. This delay is computed by using Equation 30-24.

$$d_{ap,l} = p_{ov}\ d_{t,1}\left(\frac{1}{P_L} - 1\right)\frac{P_{lt}}{1 - P_{lt} - P_{rt}}$$

Equation 30-24

where

$d_{ap,l}$ = through vehicle delay due to left turns (s/veh),

p_{ov} = probability of left-turn bay overflow (decimal),

$d_{t,1}$ = average delay to through vehicles in the inside lane (s/veh),

P_L = proportion of left-turning vehicles in the shared lane (decimal),

P_{rt} = proportion of right-turning vehicles on the subject approach (decimal), and

P_{lt} = proportion of left-turning vehicles on the subject approach (decimal).

As indicated by Equation 30-24, the delay due to left turns is based on the value of several variables. The following sequence of computations can be used to estimate these values (*5*).

Step 1: Compute the Probability of a Lane Change

Equation 30-25

$$P_{lc} = 1 - \left(\left[2 \frac{v_{app}}{s_{lc}} \right] - 1 \right)^2 \quad \geq 0.0$$

with

Equation 30-26

$$v_{app} = \frac{v_{lt} + v_{th} + v_{rt}}{N_{sl} + N_t + N_{sr}}$$

where

P_{lc} = probability of a lane change among the approach through lanes,

v_{app} = average demand flow rate per through lane (upstream of any turn bays on the approach) (veh/h/ln),

s_{lc} = maximum flow rate in which a lane change can occur = 3,600/t_{lc} (veh/h/ln),

t_{lc} = critical merge headway = 3.7 (s),

v_{lt} = left-turn demand flow rate (veh/h),

v_{th} = through demand flow rate (veh/h),

v_{rt} = right-turn demand flow rate (veh/h),

N_{sl} = number of lanes in shared left-turn and through-lane group (ln),

N_t = number of lanes in exclusive through-lane group (ln), and

N_{sr} = number of lanes in shared right-turn and through-lane group (ln).

If the ratio v_{app}/s_{lc} in Equation 30-25 exceeds 1.0, then it should be set to 1.0.

Step 2: Compute Through Vehicle Equivalent for Left-Turn Vehicle

Equation 30-27

$$E_{L1} = \frac{1{,}800}{c_l}$$

with

Equation 30-28

$$c_l = \frac{v_o \, e^{-v_o \, t_{cg} / 3{,}600}}{1 - e^{-v_o \, t_{fh} / 3{,}600}}$$

where

E_{L1} = equivalent number of through cars for a permitted left-turning vehicle;

c_l = capacity of a left-turn movement with permitted left-turn operation (veh/h);

v_o = opposing demand flow rate (veh/h);

t_{fh} = follow-up headway (4.5 if the subject left turn is served in a shared lane; 2.5 if the subject left turn is served in an exclusive lane) (s), and

t_{cg} = critical headway = 4.5 (s).

Step 3: Compute Modified Through-Vehicle Equivalent

$$E_{L1,m} = \left(E_{L1} - 1\right) P_{lc} + 1$$

Equation 30-29

$$E_{R,m} = \left(E_{R,ap} - 1\right) P_{lc} + 1$$

Equation 30-30

where

$E_{L1,m}$ = modified through-car equivalent for a permitted left-turning vehicle,

$E_{R,m}$ = modified through-car equivalent for a protected right-turning vehicle, and

$E_{R,ap}$ = equivalent number of through cars for a protected right-turning vehicle at an access point = 2.20.

Other variables are as previously defined.

Step 4: Compute Proportion of Left Turns in Inside Through Lane

$$P_L = \frac{-b + \sqrt{b^2 - 4\, I_t\, R\, c}}{2\, I_t\, R} \leq 1.0$$

Equation 30-31

with

$$b = R - P_{lt}\left\lfloor I_t + (N_{sl} + N_t + N_{sr} - 1)\,(\,[1 + I_t]E_{L1,m} - 1)\right\rfloor$$

Equation 30-32

$$c = P_{lt}\ (N_{sl} + N_t + N_{sr})$$

Equation 30-33

$$R = 1 + P_{rt}\ (E_{R,m} - 1)$$

Equation 30-34

where

R, b, c = intermediate calculation variables, and

I_t = indicator variable = 1.0 when equations are used to evaluate delay due to left turns and 0.00001 when equations are used to evaluate delay due to right turns.

Other variables are as previously defined.

If the number of through lanes on the subject intersection approach (= $N_{sl} + N_t + N_{sr}$) is equal to 1.0, then $P_L = P_{lt}$. If the right-turn vehicles are provided an exclusive right-turn lane, then P_{rt} should equal 0.0 in Equation 30-34 and in all subsequent equations.

The indicator variable I_t is used to adapt the equations to the analysis of lane volume for both left-turn- and right-turn-related delays. The variable has a value of 1.0 when applied to the evaluation of left-turn-related delays. In this situation, it models the condition in which there is one or more left-turn vehicles blocking the inside lane. In contrast, the variable has a negligibly small value when applied to right-turn-related delays. It models flow conditions in which all lanes are unblocked.

Step 5: Compute Proportion of Right Turns in Outside Through Lane

Equation 30-35

$$P_R = P_{rt}\frac{s_1/1{,}800 + N_{sl} + N_t + N_{sr} - 1}{1 - P_{rt}\left(s_1/1{,}800 + N_{sl} + N_t + N_{sr} - 2\right)\left(E_{R,m} - 1\right)} \le 1.0$$

with

Equation 30-36

$$s_1 = \frac{1{,}800\,(1 + P_L\ I_t)}{1 + P_L\ (E_{L1,m} - 1) + (P_L\ E_{L1,m}\ I_t)}$$

where s_1 is the saturation flow rate for the inside lane (veh/h/ln). If the number of through lanes on the subject intersection approach (= $N_{sl} + N_t + N_{sr}$) is equal to 1.0, then $P_R = P_{rt}$.

Step 6: Compute Inside Lane and Outside Lane Flow Rates

Equation 30-37

$$v_1 = \frac{v_{lt}}{P_L}$$

Equation 30-38

$$v_n = \begin{cases} \dfrac{v_{rt}}{P_R} & \text{if } P_R > 0.0 \\ \dfrac{v_{lt} + v_{th} + v_{rt} - v_1}{N_{sl} + N_t + N_{sr} - 1} & \text{if } P_R = 0.0 \end{cases}$$

where

v_1 = flow rate for the inside lane (veh/h/ln), and

v_n = flow rate for the outside lane (veh/h/ln).

Other variables are as previously defined.

Step 7: Compute Intermediate Lane Flow Rate

If there are more than two lanes on the subject intersection approach, then Equation 30-39 can be used to estimate the flow rate in the intermediate lanes.

Equation 30-39

$$v_i = \frac{v_{lt} + v_{th} + v_{rt} - v_1 - v_n}{N_{sl} + N_t + N_{sr} - 2}$$

where v_i is the flow rate for lane i (veh/h/ln), and other variables are as previously defined. The flow rates in lanes 2, 3, . . . , $n - 1$ are identical and equal to the value obtained from Equation 30-39.

Step 8: Compute Merge Capacity

Equation 30-40 is used to compute the merge capacity available to through drivers waiting in the inside lane of a multilane approach.

Equation 30-40

$$c_{mg} = \frac{v_2\ e^{-v_2\ t_{lc}/3{,}600}}{1 - e^{-v_2\ t_{lc}/3{,}600}}$$

where

c_{mg} = merge capacity (veh/h),

v_2 = flow rate in the adjacent through lane (veh/h/ln), and

t_{lc} = critical merge headway = 3.7 (s).

Step 9: Compute Delay to Through Vehicles That Merge

$$d_{mg} = 3{,}600\left(\frac{1}{c_{mg}} - \frac{1}{1{,}800}\right) + 900\,T\left[\frac{v_{mg}}{c_{mg}} - 1 + \sqrt{\left(\frac{v_{mg}}{c_{mg}} - 1\right)^2 + \frac{8\,v_{mg}}{c_{mg}^2 T}}\right]$$

Equation 30-41

with

$$v_{mg} = v_1 - v_{lt} \quad \geq 0.0$$

Equation 30-42

where

d_{mg} = merge delay (s/veh),

v_{mg} = merge flow rate (veh/h/ln), and

T = analysis period duration (h).

Other variables are as previously defined.

This delay is incurred by through vehicles that stop in the inside lane and eventually merge into the adjacent through lane. The "1/1,800" term included in Equation 30-41 extracts the service time for the through vehicle from the delay estimate, such that the delay estimate represents the increase in travel time resulting from the left-turn queue.

Step 10: Compute Inside Lane Capacity

Equation 30-43 is used to compute the capacity of the inside lane for vehicles that do not merge.

$$c_{nm} = \frac{1{,}800\,(1 + P_L)}{1 + P_L\,(E_{L1} - 1) + (P_L\;E_{L1})}$$

Equation 30-43

where c_{nm} is the nonmerge capacity for the inside lane (veh/h). The unadjusted through vehicle equivalent for a left-turn vehicle E_{L1} is used in this equation to estimate the nonmerge capacity. Other variables are as previously defined.

Step 11: Compute Delay to Through Vehicles That Do Not Merge

$$d_{nm} = 3{,}600\left(\frac{1}{c_{nm}} - \frac{1}{1{,}800}\right) + 900\,T\left[\frac{v_1}{c_{nm}} - 1 + \sqrt{\left(\frac{v_1}{c_{nm}} - 1\right)^2 + \frac{8\,v_1}{c_{nm}^2 T}}\right]$$

Equation 30-44

where d_{nm} is the nonmerge delay for the inside lane (s/veh), and other variables are as defined previously. This delay is incurred by through vehicles that stop in the inside lane and wait for the queue to clear. These vehicles do not merge into the adjacent lane.

Step 12: Compute Delay to Through Vehicles in the Inside Lane

This delay is estimated as the smaller of the delay relating to the merge and nonmerge maneuvers. It is computed by using Equation 30-45.

Equation 30-45

$$d_{t,1} = \min\left(d_{nm}, d_{mg}\right)$$

Step 13: Compute the Probability of Left-Turn Bay Overflow

Equation 30-46

$$p_{ov} = \left(\frac{v_{lt}}{c_l}\right)^{N_{qx,lt}+1}$$

with

Equation 30-47

$$N_{qx,lt} = \frac{N_{lt}\ L_{a,lt}}{L_h}$$

where

p_{ov} = probability of left-turn bay overflow (decimal),

$N_{qx,lt}$ = maximum queue storage for the left-turn movement (veh),

N_{lt} = number of lanes in the left-turn bay (ln),

$L_{a,lt}$ = available queue storage distance for the left-turn movement (ft/ln), and

L_h = average vehicle spacing in the stationary queue (see Equation 30-10) (ft/veh).

Other variables are as previously defined.

For an undivided cross section, the number of left-turn vehicles that can store $N_{qx,lt}$ is equal to 0.0.

Step 14: Compute Through Vehicle Delay due to Left Turns

The through vehicle delay due to left turns $d_{ap,l}$ is computed by using Equation 30-24.

DELAY DUE TO RIGHT TURNS

A vehicle turning right from the major street into an access point often delays the through vehicles that follow it. Through vehicles are delayed because they have to reduce speed to avoid a collision with the vehicle ahead, the first of which has reduced speed to avoid a collision with the right-turning vehicle. This delay can be several seconds in duration for the first few through vehicles but will always decrease to negligible values for subsequent vehicles as the need to reduce speed diminishes. For purposes of running time calculation, this delay must be averaged over all through vehicles traveling in the subject direction. The resulting average delay is computed by using Equation 30-48.

Equation 30-48

$$d_{ap,r} = 0.67\ d_{t|r}\ \frac{P_{rt}}{1 - P_{lt} - P_{rt}}$$

where

$d_{ap,r}$ = through vehicle delay due to right turns (s/veh), and

$d_{t|r}$ = through vehicle delay per right-turn maneuver (s/veh).

Other variables are as previously defined.

The variable $d_{t|r}$ in Equation 30-48 converges to 0.0 as the proportion of turning vehicles approaches 1.0. The constant 0.67 represents a calibration factor based on field data. The steps undertaken to quantify this factor are described in the remainder of this subsection. Equation 30-48 can also be used to estimate the delay due to left-turn vehicles on a one-way street. In this case, variables associated with the right-turn movement would be redefined as applicable to the left-turn movement and vice versa.

As indicated by Equation 30-48, the delay due to right turns is based on the value of several variables. The following sequence of computations can be used to estimate these values (*6*).

Step 1: Compute Minimum Speed for the First Through Vehicle

$$u_m = 1.47\ S_f - r_d\left(H_1 - h_{|\Delta<h<H_1}\right) \quad \geq u_{rt}$$

Equation 30-49

with

$$h_{|\Delta<h<H_1} = \frac{1}{\lambda} + \frac{\Delta - H_1\ e^{-\lambda(H_1-\Delta)}}{1-e^{-\lambda(H_1-\Delta)}}$$

Equation 30-50

$$H_1 = \frac{1.47 S_f - u_{rt}}{r_d} + t_{cl} + \frac{L_h}{1.47 S_f} \quad \geq \Delta$$

Equation 30-51

$$\lambda = \frac{1}{1/q_n - \Delta}$$

Equation 30-52

where

u_m = minimum speed of the first through vehicle given that it is delayed (ft/s),

u_{rt} = right-turn speed = 20 (ft/s),

S_f = free-flow speed (mi/h),

$h_{|\Delta<h<H_1}$ = average headway of those headways between Δ and H_1 (s/veh),

Δ = headway of bunched vehicle stream = 1.5 (s/veh),

H_1 = maximum headway that the first through vehicle can have and still incur delay (s/veh),

r_d = deceleration rate = 6.7 (ft/s^2),

t_{cl} = clearance time of the right-turn vehicle = 0.6 (s),

L_h = average vehicle spacing in stationary queue (see Equation 30-10) (ft/veh),

λ = flow rate parameter (veh/s),

q_n = outside lane flow rate = v_n/3,600 (veh/s), and

v_n = flow rate for the outside lane (veh/h/ln).

The right-turn speed u_{rt} used in Equation 30-49 and Equation 30-51 is likely to be sensitive to access point design, including the approach profile, throat width, and curb radius. For level profiles and nominal throat widths, the speed can vary from 15 to 25 ft/s for radii varying from 20 to 60 ft, respectively. A

default turn speed of 20 ft/s is recommended when information is not available to make a more accurate estimate.

The flow rate for the outside lane v_n is computed by using Steps 3, 4, 5, and 6 from the Delay due to Left Turns procedure described in the previous subsection. However, the probability of a lane change P_{lc} is set equal to 1.0 when the calculations in Step 3 are made. In Steps 4 and 5, the variable I_t is set equal to 0.00001. The proportion of right-turning vehicles in the shared lane P_R is also computed at this point and used in a later step.

Step 2: Compute Delay to the First Through Vehicle

Equation 30-53

$$d_1 = \frac{(1.47S_f - u_m)^2}{2\,(1.47S_f)}\left(\frac{1}{r_d} + \frac{1}{r_a}\right)$$

where d_1 = conditional delay to first through vehicle (s/veh), r_a = acceleration rate = 3.5 (ft/s²), and other variables are as previously defined.

Step 3: Compute Delay to the Second Through Vehicle

Equation 30-54

$$d_2 = d_1 - (h_{|\Delta<h<H_2} - \Delta)$$

with

Equation 30-55

$$h_{|\Delta<h<H_2} = \frac{1}{\lambda} + \frac{\Delta - H_2\ e^{-\lambda(H_2-\Delta)}}{1 - e^{-\lambda(H_2-\Delta)}}$$

Equation 30-56

$$H_2 = d_1 + \Delta$$

where d_2 is the conditional delay to Vehicle 2 (s/veh), and other variables are as previously defined.

Step 4: Compute Delay to the Third and Subsequent Through Vehicles

Equation 30-57

$$d_i = d_{i-1} - (h_{|\Delta<h<H_i} - \Delta)$$

with

Equation 30-58

$$h_{|\Delta<h<H_i} = \frac{1}{\lambda} + \frac{\Delta - H_i\ e^{-\lambda(H_i-\Delta)}}{1 - e^{-\lambda(H_i-\Delta)}}$$

Equation 30-59

$$H_i = d_{i-1} + \Delta$$

where d_i is the conditional delay to vehicle i (i = 3, 4, . . . ,) (s/veh), and other variables are as previously defined. As shown by Equation 30-54 and Equation 30-57, the delay to each subsequent through vehicle is less than or equal to that of the preceding vehicle. In fact, the sequence of delays always converges to zero when the average flow rate in the outside lane is less than 1/Δ.

Step 4 should be repeated for the third and subsequent through vehicles until the delay computed for vehicle i is less than 0.1 s. In general, this criterion results in delay being computed for only the first two or three vehicles.

Step 5: Compute Through Vehicle Delay per Right-Turn Maneuver

The through vehicle delay for the first two vehicles is computed by using Equation 30-60.

$$d_{t|r} = d_1\left(1-e^{-\lambda(H_1-\Delta)}\right)(1-P_R) + d_2\left(1-e^{-\lambda(H_1-\Delta)}\right)\left(1-e^{-\lambda(H_2-\Delta)}\right)(1-P_R)^2$$

Equation 30-60

where $d_{t|r}$ is the through vehicle delay per right-turn maneuver (s/veh), and other variables are as previously defined. If three or more vehicles are delayed, then an additional term needs to be added to Equation 30-60 for each subsequent vehicle. In this situation, Equation 30-61 can be used to compute the delay for any number of vehicles.

$$d_{t|r} = \sum_{i=1}^{\infty}\left[d_i \times \prod_{j=1}^{i}\left(1-e^{-\lambda(H_j-\Delta)}\right) \times (1-P_R)^i\right]$$

Equation 30-61

Step 6: Compute Through Vehicle Delay due to Right Turns

The through vehicle delay due to right turns $d_{ap,r}$ is computed by using Equation 30-48.

4. QUICK ESTIMATION METHOD

INTRODUCTION

This section describes a simplified method for evaluating the operation of a coordinated street segment with signalized boundary intersections. The method addresses automobile operation. It is focused on the analysis of the through movement at the boundary intersections. This method can be used when minimal data are available for the analysis and only approximate results are desired.

INPUT DATA REQUIREMENTS

The overall data requirements are summarized in Exhibit 30-6. Some of the input requirements may be met by assumed values or default values. Other data items are site-specific and must be obtained in the field. The objective of using the quick estimation method is to minimize the need for the collection of detailed field data.

Exhibit 30-6
Input Data Requirements for the Quick Estimation Method

Data Category	Location	Input Data Element
Traffic characteristics	Boundary intersection	Through-demand flow rate
		Through-saturation flow rate
		Volume-to-capacity ratio of the upstream movements
	Segment	Platoon ratio
		Midsegment flow rate
		Midsegment delay
Geometric design	Boundary intersection	Number of through lanes
		Upstream intersection width
	Segment	Number of through lanes
		Segment length
		Restrictive median length
		Nonrestrictive median length
		Proportion of segment with curb
		Number of access point approaches
Signal control	Boundary intersection	Effective green-to-cycle-length ratio
		Cycle length
Other	Segment	Analysis period duration
		Speed limit

At a minimum, the analyst must provide traffic volumes and the approach-lane configuration for the subject intersection. Default values for several variables are specifically identified in the methodology and integrated into the method. These values have been selected to be generally representative of typical conditions. Additional default values are identified in Section 3 of Chapter 17, Urban Street Segments.

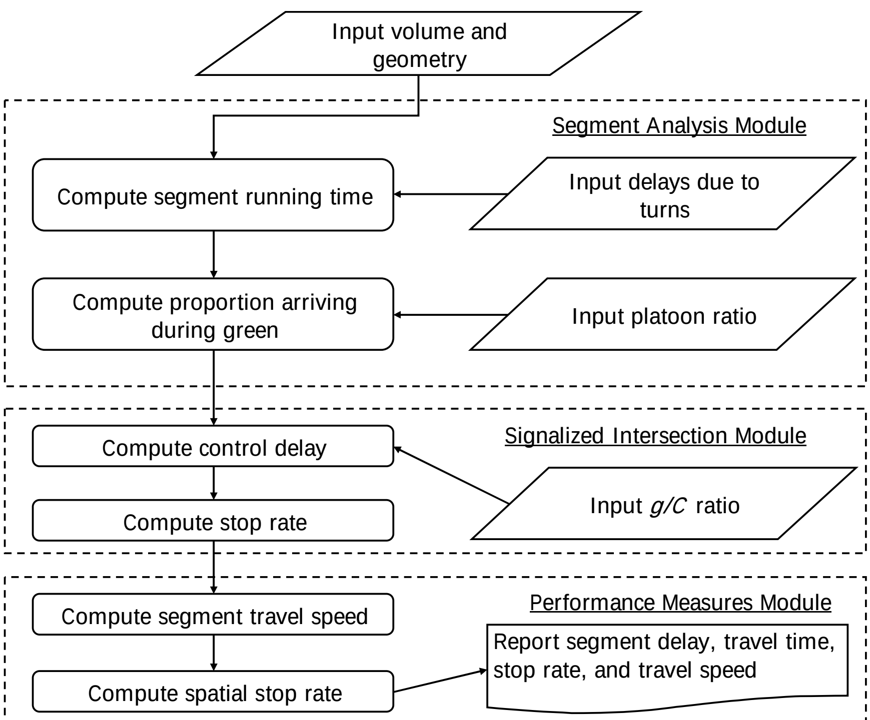
METHODOLOGY

The analysis consists of five computational steps. These steps are

- Determine running time,
- Determine proportion arriving during green,
- Determine through control delay,
- Determine through stop rate, and
- Determine travel speed and spatial stop rate.

Each step is executed in the sequence presented in the preceding list. This sequence is illustrated by the flow chart in Exhibit 30-7. The rectangles with rounded corners indicate the computational steps. The parallelograms indicate where input data are needed.

Exhibit 30-7
Quick Estimation Method for Urban Street Segments



The computations associated with each step identified in Exhibit 30-7 are described in Section 2 of Chapter 17. These computations are conveniently illustrated here in a series of worksheets; each worksheet corresponds to one or two of the calculation steps.

The first of the computational worksheets is the Running Time worksheet. It is shown as Exhibit 30-8 (values shown apply to the Example Problem, as discussed in a subsequent section).

Exhibit 30-8
Quick Estimation Running Time Worksheet

RUNNING TIME WORKSHEET				
General Information		**Site Information**		
Analyst	JME	Street	Texas Avenue	
Agency or Company	ACME Engr.	Jurisdiction		
Date Performed	9/30/10	Analysis Year	2010	
Analysis Time Period	5:30 p.m. to 5:45 p.m.	Analysis Level	planning	
Input Data				
	Segment 1		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Segment Data				
Number of through lanes for length of segment (N_{th}), ln	2	2		
Speed limit (S_{pl}), mi/h	35	35		
Midsegment volume (v_m), veh/h	1,150	1,150		
Total delay due to turns into access points (Σd_{ap}), s/veh	0.52	0.52		
Delay due to other midsegment sources (d_{other}), s/veh	0	0		
Length of segment (L), ft	1,800	1,800		
Width of upstream boundary intersection (W_i), ft	50	50		
Length of segment with restrictive median (L_{rm}), ft	0	0		
Length of segment with nonrestrictive median (L_{nr}), ft	0	0		
Start-up lost time (l_1), s	2.0	2.0		
Access Data				
Proportion of street with curb on right-hand side (p_{curb})	0.70	0.70		
Number of access points on right-hand side (N_{ap})	4	4		
Running Time Computation				
Adjusted segment length (L_{adj}), ft $L_{adj} = L - W_i$	1,750	1,750		
Proportion of segment length with restrictive median (p_{rm}) $p_{rm} = L_{rm} / L_{adj}$	0	0		
Speed constant (S_0), mi/h $S_0 = 25.6 + 0.47\ S_{pl}$	42.1	42.1		
Adjustment for cross section (f_{CS}), mi/h $f_{CS} = 1.5\ p_{rm} - 0.47\ p_{curb} - 3.7\ p_{curb}\ p_{rm}$	-0.3	-0.3		
Access point density (D_a), access points/mi $D_a = 5280\ (N_{ap,EB/NB} + N_{ap,WB/SB}) / L_{adj}$	24.1	24.1		
Adjustment for access points (f_A), mi/h $f_A = -0.078\ D_a / N_{th}$	-0.9	-0.9		
Base free-flow speed (S_{fo}), mi/h $S_{fo} = S_0 + f_{CS} + f_A$	40.8	40.8		
Segment length adjustment factor (f_L) $f_L = 1.02 - 4.7\ (S_{fo} - 19.5)/\max(L, 400) \le 1.0$	0.96	0.96		
Free-flow speed (S_f), mi/h $S_f = S_{fo}\ f_L$	39.3	39.3		
Proximity adjustment factor (f_v) $f_v = \dfrac{2}{1 + \left(1 - \dfrac{v_m}{52.8\ N_{th}\ S_f}\right)^{0.21}}$	1.03	1.03		
Running time (t_R), s $t_R = \dfrac{6.0 - l_1}{0.0025\ L} + \dfrac{3{,}600\ L}{5{,}280\ S_f} f_v + \sum d_{ap} + d_{other}$	33.7	33.7		

Note: The first term in the running time equation is only applicable to segments with signalized or STOP- or YIELD-controlled boundary intersections.

The Running Time worksheet combines input data describing the segment geometric design, speed limit, volume, and access point frequency to estimate the base free-flow speed. This speed is then adjusted for segment length effects to obtain the expected free-flow speed. The free-flow speed is then used to estimate a free-flow travel time, which is adjusted for the proximity of other vehicles. Delay that is caused by turns into access points or other sources is added to the adjusted travel time. Default values for the delay due to turns at midsegment access points are listed in Exhibit 17-13 in Chapter 17. These defaults can be used when other, more accurate estimates of this delay are not available. The result of these adjustments is an estimate of the expected segment running time.

The second of the computational worksheets is the Proportion Arriving During Green worksheet. It is shown as Exhibit 30-9. This worksheet is designed for the analysis of the segment through-lane group. It documents the calculation of the proportion of vehicles that arrive during the green indication. Input data include the effective green-to-cycle-length ratio and platoon ratio.

Exhibit 30-9
Quick Estimation Proportion Arriving During Green Worksheet

PROPORTION ARRIVING DURING GREEN WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
	Segment 1		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Signal Timing Data				
Effective green-to-cycle-length ratio (g/C)	0.47	0.47		
Traffic Data				
Platoon ratio (R_p)	1.43	0.67		
Proportion Arriving During Green Computation				
Proportion arriving during green (P) $P = R_p\ (g/C)$	0.67	0.31		

The third computational worksheet is the Control Delay worksheet. It is shown as Exhibit 30-10. This worksheet is designed for the analysis of the segment through-lane group. Input variables include the analysis period duration, cycle length, effective green-to-cycle-length ratio, volume, saturation flow rate, and lanes. The proportion of arrivals during green is obtained from the previous worksheet.

The control delay is computed as the sum of two components. The first component to be computed is the uniform delay. The notation "min(1, X)" is shown in the equation used to compute this delay. It means that the value to be substituted for this text is the smaller of 1.0 and the volume-to-capacity ratio.

Exhibit 30-10
Quick Estimation Control Delay Worksheet

CONTROL DELAY WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
Analysis period (T), h: 0.25	**Segment 1**		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Signal Timing Data				
Cycle length (C), s	100	100		
Effective green-to-cycle-length ratio (g/C)	0.47	0.47		
Traffic Data				
Through-lane group volume (v_{th}), veh/h	968	950		
Lane group saturation flow rate (s), veh/h/ln	1,800	1,800		
Proportion of arrivals during green (P)	0.67	0.31		
Volume-to-capacity ratio (X_u) of the upstream movements	0.57	0.57		
Geometric Design Data				
Number of through lanes (N_{th}), ln	2	2		
Delay Computation				
Capacity (c), veh/h $c = N_{th}\ s\ g/C$	1,692	1,692		
Volume-to-capacity ratio (X) $X = v_{th}/c$	0.57	0.56		
Uniform delay (d_1), s/veh $d_1 = \dfrac{0.5\,C\,(1 - g/C)^2}{1 - [\min(1, X)\,g/C]}$	19.2	19.1		
Upstream filtering adjustment factor (I) $I = 1.0 - 0.91\,X_u^{2.68} \geq 0.090$	0.80	0.80		
Incremental delay (d_2), s/veh $d_2 = 900\,T\left[(X-1) + \sqrt{(X-1)^2 + \dfrac{4\,I\,X}{c\,T}}\right]$	1.13	1.08		
Progression adjustment factor (PF) $PF = (1 - P)/(1 - g/C)$	0.62	1.30		
Control delay (d), s/veh $d = d_1\,(PF) + d_2$	13.0	25.8		

The second delay component is the incremental delay, which is based on the upstream filtering adjustment factor. This factor requires the variable X_u. This variable can be estimated as the volume-to-capacity ratio of the segment through-lane group at the upstream signalized intersection. Additional detail on the calculation of this ratio for long segments is provided in Section 1 of Chapter 18, Signalized Intersections.

The fourth computational worksheet is the Stop Rate worksheet. It is shown as Exhibit 30-11. This worksheet is designed for the analysis of the segment through-lane group. The input variables are the same as those needed for the Control Delay worksheet with the addition of speed limit. The average speed during the analysis period is estimated by using the equation provided. If the average speed is known, it should be substituted for the estimated value.

Exhibit 30-11
Quick Estimation Stop Rate Worksheet

STOP RATE WORKSHEET				
General Information				
Project Description: Texas Avenue, 5:30 p.m. to 5:45 p.m.				
Input Data				
Analysis period (T), h: 0.25	**Segment 1**		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Signal Timing Data				
Cycle length (C), s	100	100		
Effective green-to-cycle-length ratio (g/C)	0.47	0.47		
Traffic Data				
Through-lane group volume (v_{th}), veh/h	968	950		
Lane group saturation flow rate (s), veh/h/ln	1,800	1,800		
Proportion of arrivals during green (P)	0.67	0.31		
Speed limit (S_{pl}), mi/h	35	35		
Incremental delay (d_2), s/veh	1.13	1.08		
Geometric Design Data				
Number of through lanes (N_{th}), ln	2	2		
Stop Rate Computation				
Effective green time (g), s $g = C\,(g/C)$	47	47		
Effective red time (r), s $r = C - g$	53	53		
Capacity (c), veh/h $c = N_{th}\, s\, g/C$	1,692	1,692		
Volume-to-capacity ratio (X) $X = v_{th}/c$	0.57	0.56		
Average speed (S_a), mi/h $S_a = 0.90\,(25.6 + 0.47\, S_{pl})$	39.9	39.9		
Threshold acceleration–deceleration delay, s $(1 - P)\, gX$	8.8	18.1		
Acceleration–deceleration delay (d_a), s $d_a = 0.393\,(S_a - 5.0)^2/S_a$	12.0	12.0		
Deterministic stop rate (h_1), stops/veh $h_1 = \frac{1 - P(1 + d_a/g)}{1 - PX}$: *if* $d_a \le (1-P)gX$; $h_1 = \frac{(1-P)(r - d_a)}{r - (1-P)gX}$: *if* $d_a > (1-P)gX$	0.30	0.74		
Second-term back-of-queue size (Q_2), veh/ln $Q_2 = c\, d_2/(3{,}600\, N_{th})$	0.26	0.25		
Full stop rate (h), stops/veh $h = h_1 + 3{,}600\, N_{th}\, Q_2/(v_{th}\, C)$	0.32	0.76		

The stop rate is computed as the sum of two components. The first component to be computed is the deterministic stop rate. Two equations are available for this computation. The correct equation to use is based on a check of the acceleration–deceleration delay relative to the computed threshold value.

The second stop rate component is based on the second-term back-of-queue size. This queue represents the average number of vehicles that are unserved at the end of the green interval. It is based on the incremental delay computed for the Control Delay worksheet.

The fifth computational worksheet is the Travel Speed and Spatial Stop Rate worksheet. It is shown as Exhibit 30-12. This worksheet is designed for the analysis of the segment through-lane group. The input values include segment

length and the full stop rate associated with other midsegment events (e.g., turns at access points). The other input data listed represent computed values and are obtained from the previous worksheets.

Exhibit 30-12
Quick Estimation Travel Speed and Spatial Stop Rate Worksheet

TRAVEL SPEED AND SPATIAL STOP RATE WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
	Segment 1		**Segment 2**	
Direction of travel	**EB/NB**	**WB/SB**	**EB/NB**	**WB/SB**
Length of segment (L), ft	1,800	1,800		
Base free-flow speed (S_{fo}), mi/h	40.8	40.8		
Running time (t_R), s	33.7	33.7		
Control delay (d), s/veh	13.0	25.8		
Full stop rate (h), stops/veh	0.32	0.76		
Full stop rate due to other midsegment sources (h_{other}), stops/veh	0	0		
Travel Speed Computation				
Travel time (T_T), s $T_T = t_R + d$	46.7	59.5		
Travel speed ($S_{T,seg}$), mi/h $S_{T,seg} = \frac{3{,}600\,L}{5{,}280\,T_T}$	26.3	20.6		
Spatial Stop Rate Computation				
Total stop rate (h_T), stops/veh $h_T = h + h_{other}$	0.32	0.76		
Spatial stop rate (H_{seg}), stops/mi $H_{seg} = \frac{5{,}280\,h_T}{L}$	0.94	2.22		
Level of Service Computation				
Travel speed as a percentage of base free-flow speed (R) $R = 100\ S_{T,seg} / S_{fo}$	64.5	50.6		
Level of service (Exhibit 17-1)	C	C		

EXAMPLE PROBLEM

The Urban Street Segment

The total length of an undivided urban street segment is 1,800 ft. It is shown in Exhibit 30-13. Both of the boundary intersections are signalized. The street has a four-lane cross section with two lanes in each direction. There are left-turn bays on the subject segment at each signalized intersection.

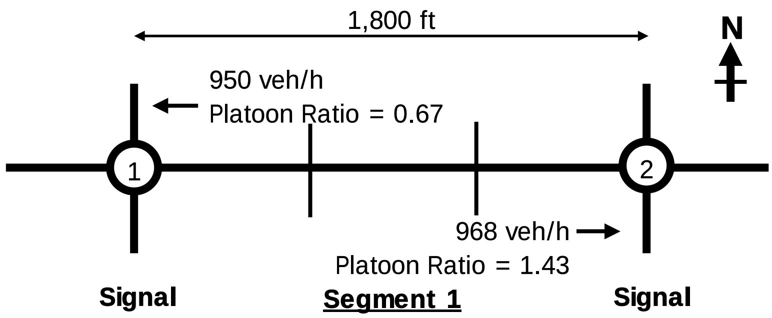


Exhibit 30-13
Quick Estimation Example Problem

The segment has two access point intersections. Each intersection has two STOP-controlled side-street approaches. The segment has some additional driveways on each side of the street; however, their turn movement volumes are too low during the analysis period for them to be considered "active."

The Question

What are the travel speed, spatial stop rate, and level of service (LOS) during the analysis hour for through automobile traffic in both directions of travel along the segment?

The Facts

Some details of the segment are shown in Exhibit 30-13. Both boundary intersections are signalized. The following additional information is known about the street segment:

Through saturation flow rate: 1,800 veh/h/ln

Midsegment volume: 1,150 veh/h

Midsegment delay: 0.52 s/veh

Number of through lanes at boundary intersection: 2

Upstream intersection width: 50 ft

Number of through lanes on segment: 2

Proportion of street with curb: 0.70

g/*C* ratio: 0.47

Cycle length: 100 s

Analysis period: 0.25 h

Speed limit: 35 mi/h

Percent left turns at access points: 6%

Percent right turns at access points: 8%

Selected Calculations

1. Compute total delay due to turns into access points	Midsegment lanes = 2 lanes Midsegment lane volume = 575 veh/h/ln Interpolate in Exhibit 17-13 to obtain 0.37 s/veh/pt through vehicle delay. Number of active access points = 2 Percent turns = 7% [= (6 + 8)/2] Total delay per access pt. = 7/10 × 0.37 = 0.26 s/veh/pt Total delay per segment = 2 × 0.26 = 0.52 s/veh
2. Compute upstream filtering factor	No information was available about the volume-to-capacity ratio for the upstream movements, so this ratio was estimated to equal the volume-to-capacity ratio for the subject movement.

Results

The calculations are shown in Exhibit 30-8 to Exhibit 30-12. The travel speed for the eastbound direction is 26.3 mi/h. The travel speed for the westbound direction is 20.6 mi/h. The eastbound and westbound spatial stop rates are 0.94 and 2.22 stops/mi, respectively. The travel speed for the eastbound direction is 64.5 percent (= 26.3/40.8 × 100) of the base free-flow speed and, according to Exhibit 17-2, corresponds to LOS C. The westbound LOS is similarly computed to be C.

5. FIELD MEASUREMENT TECHNIQUES

This section describes two techniques for estimating key vehicular traffic characteristics by using field data. The first technique is used to estimate free-flow speed. The second technique is used to estimate average travel speed.

FREE-FLOW SPEED

The following steps can be used to determine the free-flow speed for vehicular traffic on an urban street segment.

Step 1. Conduct a spot-speed study at a midsegment location during low-volume conditions. Record the speed of 100 or more free-flowing passenger cars. A car is free-flowing when it has a headway of 8 s or more to the vehicle ahead and 5 s or more to the vehicle behind in the same traffic lane.

Step 2. Compute the average of the spot speeds S_{spot} and their standard deviation σ_{spot}.

Step 3. Compute the segment free-flow speed S_f as a space mean speed by using Equation 30-62.

$$S_f = S_{spot} - \frac{\sigma_{spot}^2}{S_{spot}}$$

Equation 30-62

where

S_f = free-flow speed (mi/h),

S_{spot} = average spot speed (mi/h), and

σ_{spot} = standard deviation of spot speeds (mi/h).

Step 4. If the base free-flow speed S_{fo} is also desired, it can be computed by using Equation 30-63.

$$S_{fo} = \frac{S_f}{f_L}$$

Equation 30-63

with

$$f_L = 1.02 - 4.7 \frac{S_f - 19.5}{\max(L_s, 400)} \quad \leq 1.0$$

Equation 30-64

where

S_{fo} = base free-flow speed (mi/h),

S_f = free-flow speed (mi/h),

L_s = distance between adjacent signalized intersections (ft), and

f_L = signal spacing adjustment factor.

Equation 30-64 was originally derived to use the base free-flow speed S_{fo} in the numerator of its second term. However, it is sufficient for this application to substitute the free-flow speed S_f.

Equation 30-64 was derived by using signalized boundary intersections. For more general applications, the definition of distance L_s is broadened such that it equals the distance between the two intersections that (*a*) bracket the subject segment and (*b*) each have a type of control that can impose on the subject through movement a legal requirement to stop or yield.

AVERAGE TRAVEL SPEED

The following steps can be used to determine the average travel speed for vehicular traffic on an urban street segment.

Step 1. Identify the time of the day (e.g., morning peak, evening peak, off-peak) during which the study will be conducted. Identify the segments to be evaluated.

Step 2. Conduct the test-car travel time study for the identified segments during the identified study period. The following factors should be considered before, or during, the field study.

- The number of travel time runs to be conducted will depend on the range of speeds found on the street. Six to 12 runs for each traffic-volume condition are typically adequate. The analyst should determine the minimum number of runs on the basis of guidance provided elsewhere (*7*).
- The objective of the data collection is to obtain the information identified in the Travel Time Field Worksheet (i.e., vehicle location and arrival and departure times at each boundary intersection). This worksheet is shown in Exhibit 30-14. In general, each row of this worksheet represents the data for one direction of travel on one segment. If the street serves traffic in two travel directions, separate worksheets are typically used to record the data for each direction of travel.
- The equipment used to record the data may include a global positioning system–equipped laptop computer or simply a pair of stopwatches. If available, an instrumented test car should be used to reduce labor requirements and to facilitate recording and analysis.
- During the test run, the average-car technique is typically used and requires that the test car travel at the average speed of the traffic stream, as judged by its driver (*7*).
- The cumulative travel time is recorded as the vehicle passes the center of each boundary intersection. Whenever the test car stops or slows (i.e., 5 mi/h or less), the observer uses a second stopwatch to measure the duration of time the vehicle is stopped or slowed. This duration (and the cause of the delay) is recorded on the worksheet on the same row that is associated with the next boundary intersection to be reached. The rows are intentionally tall so that a midsegment delay and the signal delay can both be recorded in the same cell.
- Test-car runs should begin at different time points in the signal cycle to avoid having all runs start from a "first in platoon" position.

- Some midsegment speedometer readings should also be recorded to check on unimpeded travel speeds and to see how they relate to the estimated free-flow speed.

Exhibit 30-14
Travel Time Field Worksheet

TRAVEL TIME FIELD WORKSHEET

General Information		**Site Information**	
Analyst	____________	Street	____________
Agency or Company	____________	Jurisdiction	____________
Date Performed	____________	Analysis Year	____________
Analysis Time Period	____________	Direction of Travel	____________

Field Data

Location (typically a boundary intersection)	Run Number: ____________			Run Number: ____________		
	Cumulative Travel Time at Location (s)	Delays due to Slow or Stop		Cumulative Travel Time at Location (s)	Delays due to Slow or Stop	
		Cause [a]	Delay Time (s)		Cause [a]	Delay Time (s)

Note: [a] Cause of delay: Ts - signal; Lt - left turn; Pd - pedestrian; Pk - parking; Ss - STOP sign; Ys - YIELD sign.

Step 3. The cumulative travel time observations between adjacent boundary intersections are subtracted to obtain the travel time for the corresponding segment. This travel time can be averaged for all test runs to obtain an average segment travel time. This average is then divided into the segment length to obtain an estimate of the average travel speed. This speed should be computed for each direction of travel for the segment.

The data should be summarized to provide the following statistics for each segment travel direction: average travel speed, average delay time for the boundary intersection, and average delay time for other sources (pedestrian, parking maneuver, etc.).

The average segment travel time for each of several consecutive segments in a common direction of travel can be added to obtain the total travel time for the facility. This total travel time can then be divided into the facility length (i.e., the total length of all segments) to obtain the average travel speed for the facility. This calculation should be repeated to obtain the average travel speed for the other direction of travel.

6. COMPUTATIONAL ENGINE DOCUMENTATION

This section uses a series of flowcharts and linkage lists to document the logic flow for the computational engine.

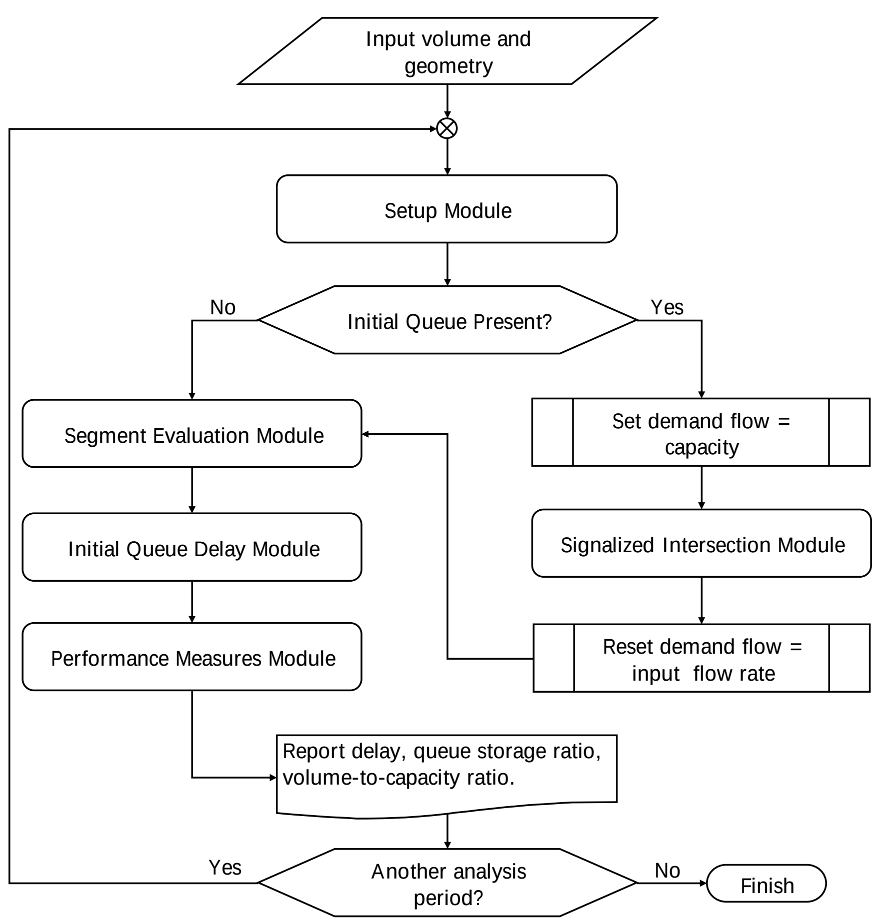
FLOWCHARTS

The methodology flowchart is shown in Exhibit 30-15. The methodology is shown to consist of five main modules:

- Setup Module,
- Segment Evaluation Module,
- Segment Analysis Module,
- Delay due to Turns Module, and
- Performance Measures Module.

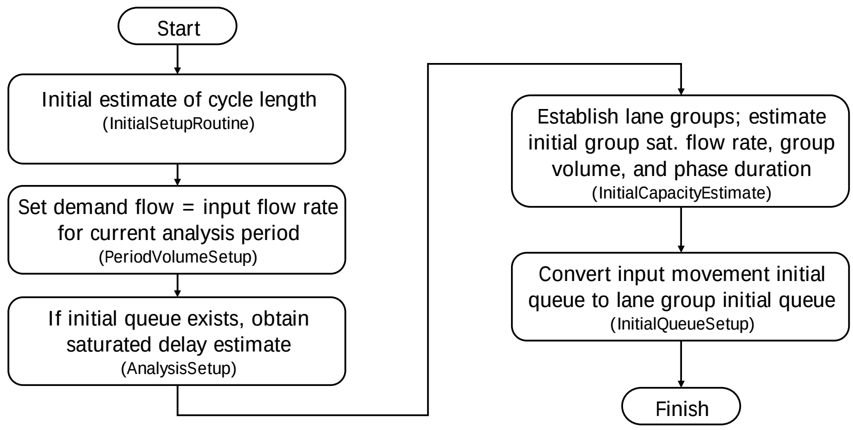
This subsection provides a separate flowchart for each of these modules.

Exhibit 30-15
Methodology Flowchart



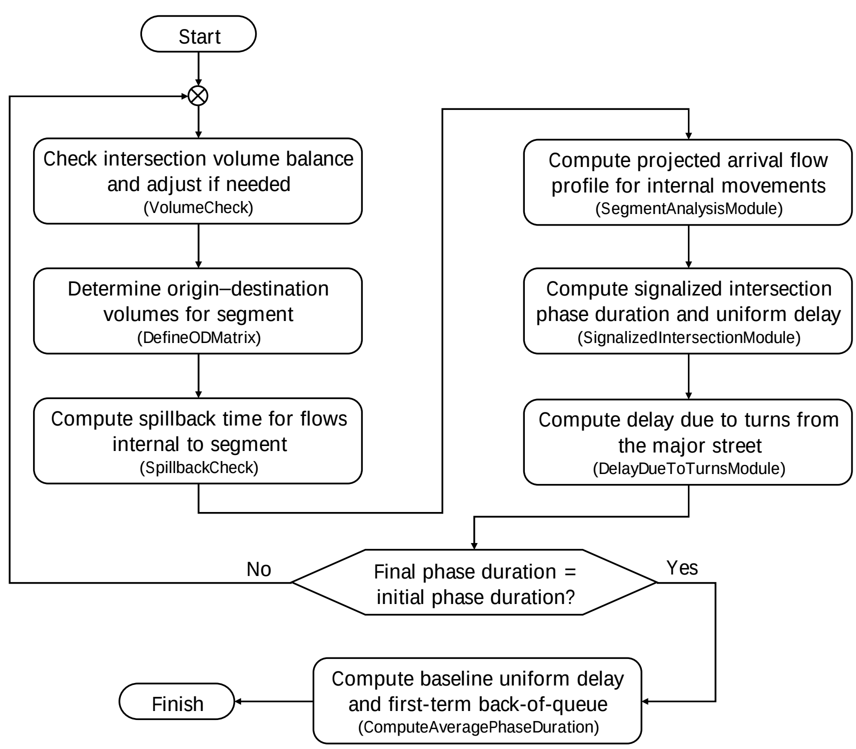
The Setup Module is shown in Exhibit 30-16. This module consists of four main routines, as shown in the large rectangles of the exhibit. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines and the Initial Queue Delay Module are described in Section 7 of Chapter 31, Signalized Intersections: Supplemental.

Exhibit 30-16
Setup Module



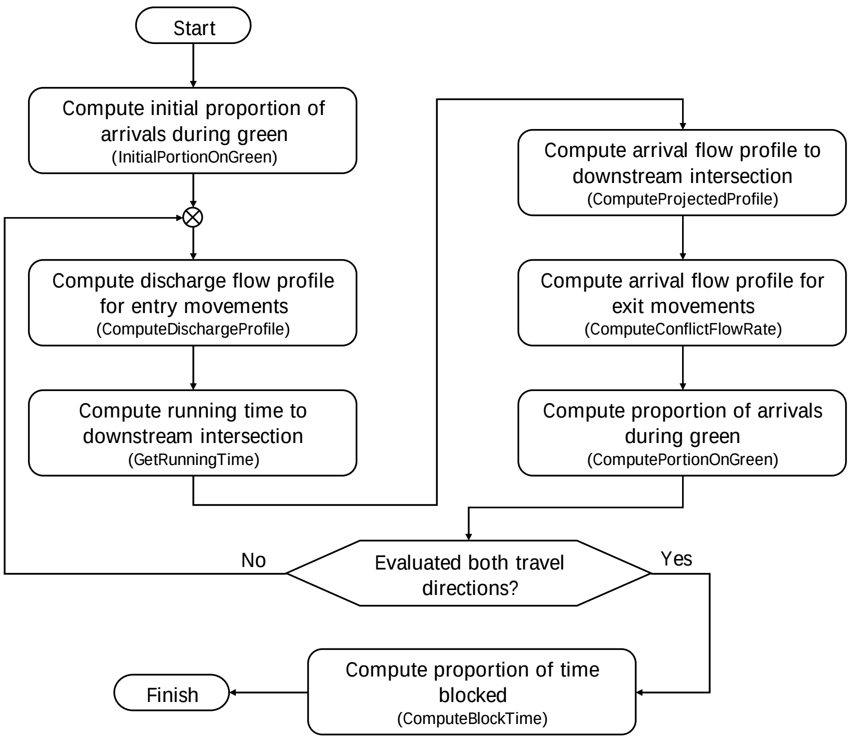
The Segment Evaluation Module is shown in Exhibit 30-17. This module consists of seven main routines. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. The Segment Analysis Module and the Delay due to Turns Module are outlined in the next two exhibits. The Signalized Intersection Module and the Compute Average Phase Duration routine are described in Section 7 of Chapter 31. The Volume Check, Define Origin–Destination Matrix, and Spillback Check routines are described further in the next subsection.

Exhibit 30-17
Segment Evaluation Module



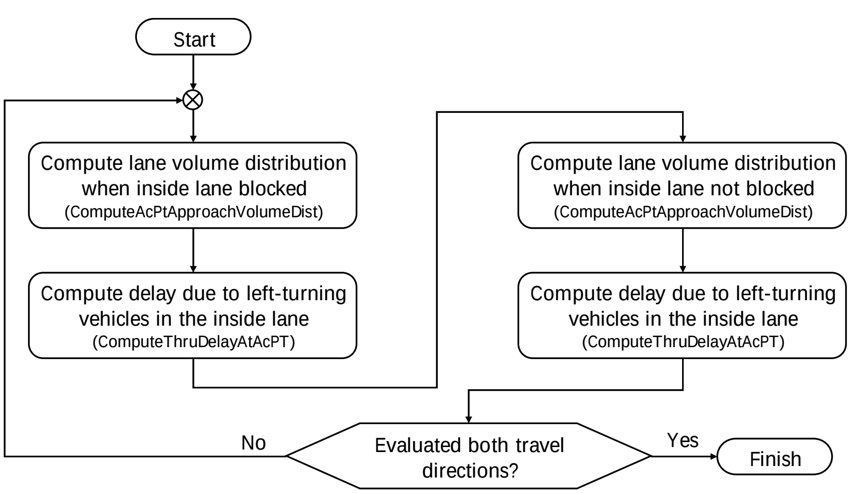
The Segment Analysis Module is shown in Exhibit 30-18. This module consists of seven main routines, six of which are implemented for both segment travel directions. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines are described further in the next subsection.

Exhibit 30-18
Segment Analysis Module



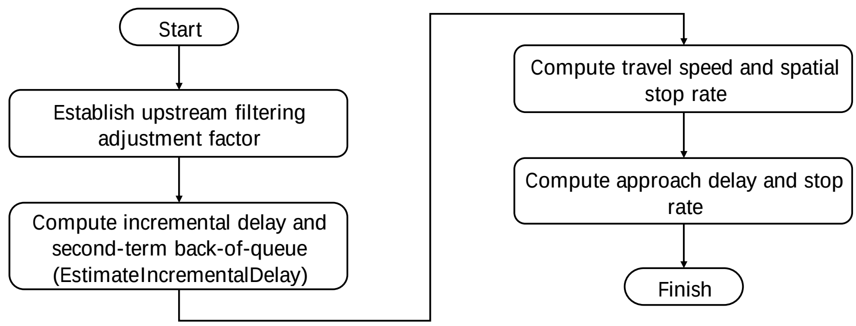
The Delay due to Turns Module is shown in Exhibit 30-19. This module consists of two main routines, each of which is implemented for both segment travel directions. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines are described further in the next subsection.

Exhibit 30-19
Delay due to Turns Module



The Performance Measures Module is shown in Exhibit 30-20. This module consists of four routines. The main function of each routine is also shown in the exhibit. One of the routines (i.e., Estimate Incremental Delay) is complicated enough to justify its development as a separate entity in the computational engine. This routine is described in Section 7 of Chapter 31, Signalized Intersections: Supplemental.

Exhibit 30-20
Performance Measures Module



LINKAGE LISTS

This subsection uses linkage lists to describe the main routines that make up the computational engine. Each list is provided in a table that identifies the routine and the various subroutines that it references. Conditions for which the subroutine is used are also provided.

The lists are organized by module, as described in the previous subsection. A total of three tables are provided to address the following three modules:

- Segment Evaluation Module,
- Segment Analysis Module, and
- Delay due to Turns Module.

The linkage list for the Segment Evaluation Module is provided in Exhibit 30-21. The main routines are listed in Column 1 and were previously identified in Exhibit 30-17.

The linkage list for the Segment Analysis Module is provided in Exhibit 30-22. The main routines are listed in Column 1 and were previously identified in Exhibit 30-18.

Finally, the linkage list for the Delay due to Turns Module is provided in Exhibit 30-23. The main routines are listed in Column 1 and were previously identified in Exhibit 30-19.

Exhibit 30-21
Segment Evaluation Module Routines

Routine	Subroutine	Conditions for Use
VolumeCheck	Ensure that discharge volume for each entry movement does not exceed its capacity.	Apply for both segment travel directions.
DefineODMatrix	ComputeODs (compute origin–destination volume for movements that enter and exit segment).	Apply to all intersections on segment and for both segment travel directions.
SpillbackCheck	ComputeSpillbackTime (compute spillback time for each exit movement at the downstream boundary intersection).	Apply for both segment travel directions.
SegmentAnalysisModule	See Exhibit 30-22.	
SignalizedIntersectionModule	See Section 7 of Chapter 31.	--
DelayDueToTurnsModule	See Exhibit 30-23.	
ComputeAveragePhaseDuration	See Section 7 of Chapter 31.	--

Exhibit 30-22
Segment Analysis Module Routines

Routine	Subroutine	Conditions for Use
InitialPortionOnGreen	Compute proportion of arrivals during green (*P*) based on current signal timing.	None
ComputeDischargeProfile	Compute discharge flow rate for each 1-s interval of signal cycle at upstream boundary intersection.	Apply to each upstream boundary intersection movement that enters segment.
GetRunningTime	Compute running time on length of street between upstream boundary intersection and subject downstream intersection.	Apply to all intersections on the segment and for both segment travel directions.
ComputeProjectedProfile	Compute arrival flow profile reflecting dispersion of platoons formed at upstream boundary intersection.	Apply to each upstream boundary intersection movement that enters segment.
ComputeConflictRate	Use arrival flow profile and origin–destination matrix to compute arrival flow rate for movements at subject intersection.	Apply to all intersections on the segment and for both segment travel directions.
	Compute conflicting flow rate at access point intersections on basis of the projected arrivals at each intersection.	Apply to all access point intersections and for both segment travel directions.
ComputePortionOnGreen	For each exit movement, compute count of vehicles arriving at downstream boundary intersection during green.	Apply to each downstream boundary intersection.
ComputeBlockTime	Use computed conflicting flow rates at each access point intersection to compute the proportion of time blocked for each nonpriority movement.	Apply to all access point intersections and for both travel segment travel directions.

Exhibit 30-23
Delay due to Turns Module Routines

Routine	Subroutine	Conditions for Use
ComputeAcPtApproach-VolumeDist	Compute the volume for each lane on the approach to the access point intersection when blocked by a left-turning vehicle.	Apply lane volume routine for case in which inside lane is blocked by a turning vehicle. Apply to all access point intersections and for both segment travel directions.
	Compute the volume for each lane on the approach to the access point intersection when *not* blocked by a left-turning vehicle.	Apply lane volume routine for case in which inside lane is *not* blocked by a turning vehicle. Apply to all access point intersections and for both segment travel directions.
ComputeThruDelayAtAcPT	Compute the probability of left-turn bay overflow at access point intersection.	If segment is undivided then probability of bay overflow is 1.0.
	Compute the delay to through movements due to a left turn at an access point. Based on lane volume estimate for case in which inside lane is blocked by a turning vehicle.	Apply to all access point intersections and for both segment travel directions.
	Compute the delay to through movements due to a right turn at an access point. Based on lane volume estimate for case in which inside lane is *not* blocked by a turning vehicle.	Apply to all access point intersections and for both segment travel directions.

7. REFERENCES

1. Van Zuylen, H. The Estimation of Turning Flows on a Junction. *Traffic Engineering and Control*, Vol. 20, No. 11, 1979, pp. 539–541.
2. Wallace, C., K. Courage, M. Hadi, and A. Gan. *TRANSYT-7F User's Guide, Vol. 4 in a Series: Methodology for Optimizing Signal Timing*. University of Florida, Gainesville, March 1998.
3. Robertson, D. *TRANSYT: A Traffic Network Study Tool*. RRL Report LR 253. Road Research Laboratory, Crowthorne, Berkshire, United Kingdom, 1969.
4. Bonneson, J., M. Pratt, and M. Vandehey. *Predicting the Performance of Automobile Traffic on Urban Streets: Final Report*. NCHRP Project 3-79. Texas Transportation Institute, Texas A&M University, College Station, Jan. 2008.
5. Bonneson, J., and J. Fitts. Delay to Major Street Through Vehicles at Two-Way Stop-Controlled Intersections. *Transportation Research: Part A—Policy and Practice*, Vol. 33, No. 3–4, 1999, pp. 237–254.
6. Bonneson, J. Delay to Major Street Through Vehicles due to Right-Turn Activity. *Transportation Research: Part A—Policy and Practice*, Vol. 32, No. 2, 1998, pp. 139–148.
7. Robertson, H., J. Hummer, and D. Nelson. *Manual of Transportation Engineering Studies*. Institute of Transportation Engineers, Washington, D.C., 2000.

CHAPTER 31
SIGNALIZED INTERSECTIONS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. TRAFFIC SIGNAL CONCEPTS

TYPES OF TRAFFIC SIGNAL CONTROL

In general, two types of traffic signal controller unit are in use today. They are broadly categorized as pretimed or actuated according to the type of control they provide. These two types of control are described as follows:

- *Pretimed control* consists of a fixed sequence of phases that are displayed in repetitive order. The duration of each phase is fixed. However, the green interval duration can be changed by time of day or week to accommodate traffic variations. The combination of a fixed phase sequence and duration produces a constant cycle length.
- *Actuated control* consists of a defined phase sequence in which the presentation of each phase depends on whether the phase is on recall or the associated traffic movement has submitted a call for service through a detector. The green interval duration is determined by the traffic demand information obtained from the detector, subject to preset minimum and maximum limits. The termination of an actuated phase requires a call for service from a conflicting traffic movement. An actuated phase may be skipped if no demand is detected.

Most modern controller units have solid-state components that are sufficiently flexible to provide either actuated control or an equivalent pretimed control (through selection of specific settings).

The operation of a pretimed controller can be described as coordinated or not coordinated. In contrast, the operation of an actuated controller can be described as fully actuated, semiactuated, or coordinated-actuated. These actuated control variations are described as follows:

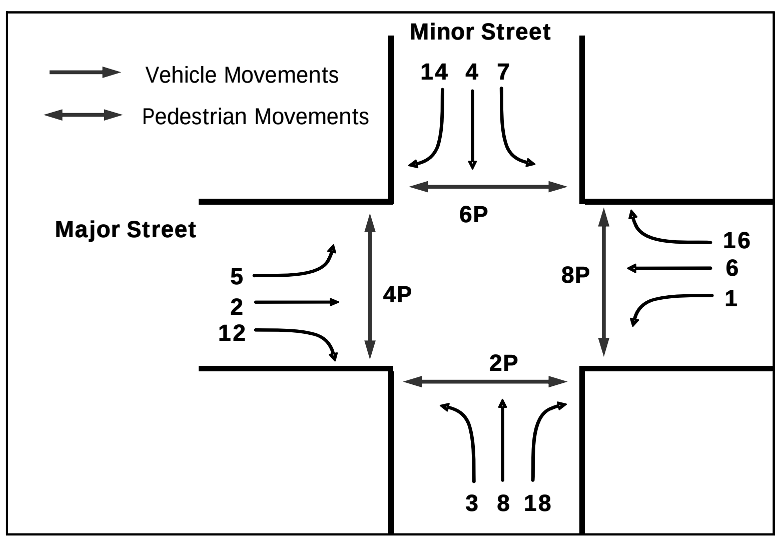
- *Fully actuated control* implies that all phases are actuated and all intersection traffic movements are detected. The sequence and duration of each phase are determined by traffic demand. Hence, this type of control is *not* associated with a constant cycle length.
- *Semiactuated control* uses actuated phases to serve the minor movements at an intersection. Only these minor movements have detection. The phases associated with the major movements are operated as “nonactuated.” The controller is programmed to dwell with the nonactuated phases displaying green for at least a specified minimum duration. The sequence and duration of each actuated phase are determined by traffic demand. Hence, this type of control is *not* associated with a constant cycle length.
- *Coordinated-actuated control* is a variation of semiactuated operation. It uses the controller’s force-off settings to constrain the noncoordinated phases associated with the minor movements so that the coordinated phases are served at the appropriate time during the signal cycle and progression for the major movements is maintained. This type of control *is* associated with a constant cycle length.

Signalized intersections that are close to one another on the same street are often operated as a coordinated signal system, in which specific phases at each intersection are operated on a common time schedule to permit the continuous flow of the associated movements at a planned speed. The signals in a coordinated system typically operate by using pretimed or coordinated-actuated control, and the coordinated phases typically serve the major-street through movements. Signalized intersections that are not part of a coordinated system are characterized as "isolated" and typically operate by using fully actuated or semiactuated control.

INTERSECTION TRAFFIC MOVEMENTS

Exhibit 31-1 illustrates typical vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. Each movement is assigned a unique number or a number and letter combination. The letter P denotes a pedestrian movement. The number assigned to each left-turn and through movement is the same as the number typically assigned to each phase by National Electrical Manufacturers Association specification.

Exhibit 31-1
Intersection Traffic Movements and Numbering Scheme



Intersection traffic movements are assigned the right-of-way by the signal controller. Each movement is assigned to one or more signal phases. A phase is defined as the green, yellow change, and red clearance intervals in a cycle that are assigned to a specified traffic movement (or movements) (*1*). The assignment of movements to phases varies in practice, depending on the desired phase sequence and the movements present at the intersection.

SIGNAL PHASE SEQUENCE

Modern actuated controllers implement signal phasing by using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other. Early controllers used a single-ring structure in which all nonconflicting movements were assigned to a common phase, and its duration was dictated by

the movement needing the most time. Of the two structures, the dual-ring structure is more efficient because it allows the controller to adapt phase duration and sequence to the needs of the individual movements. The dual-ring structure is typically used with eight phases; however, more phases are available for complex signal phasing. The eight-phase dual-ring structure is shown in Exhibit 31-2. The symbol Φ represents the word "phase," and the number following the symbol represents the phase number.

Exhibit 31-2
Dual-Ring Structure with Illustrative Movement Assignments

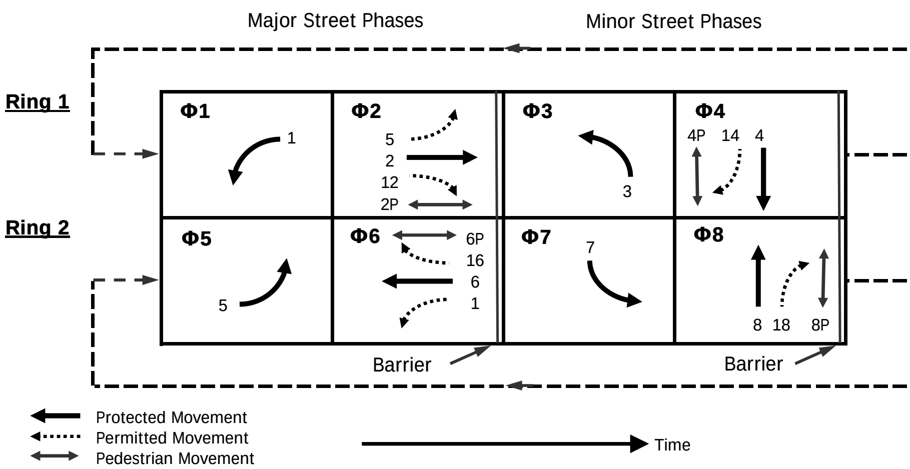


Exhibit 31-2 shows one way that traffic movements can be assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is assigned to an exclusive phase. During this phase, the left-turn movement is "protected" so that it receives a green arrow indication. Each through, right-turn, and pedestrian movement combination is also assigned to an exclusive phase. The dashed arrows indicate turn movements that are served in a "permitted" manner so that the turn can be completed only after yielding the right-of-way to conflicting movements.

Two rings and two barriers are identified in Exhibit 31-2. A ring consists of two or more sequentially timed conflicting phases. Ring 1 consists of Phases 1, 2, 3, and 4. Ring 2 consists of Phases 5, 6, 7, and 8. A barrier is used when there are two or more rings. It represents a reference point in the cycle where one phase in each ring must reach a common point of termination. In Exhibit 31-2, a barrier is shown following Phases 2 and 6. A second barrier is shown following Phases 4 and 8. Between barriers, only one phase can be active at a time in each ring.

The ring structure dictates the sequence of phase presentation. Some common rules are provided in the following list:

- Phase Pairs 1–2, 3–4, 5–6, and 7–8 typically occur in sequence. Thus, Phase Pair 1–2 begins with Phase 1 and ends with Phase 2. Within each phase pair, it is possible to reverse the order of the pair. Thus, the Pair 1–2 could be set to begin with Phase 2 and end with Phase 1 if it is desired to have the left-turn Phase 1 lag through Phase 2.
- Phase Pair 1–2 can operate concurrently with Phase Pair 5–6. That is, Phase 1 or 2 can time with Phase 5 or 6. Similarly, Phase Pair 3–4 can

operate concurrently with Phase Pair 7–8. These phase pairs are also known as concurrency groups.

- For a given concurrency group, the last phase to occur in one phase pair must end at the same time as the last phase to occur in the other pair (i.e., end together at the barrier).
- Phases between two barriers are typically assigned to the movements on a common street.

OPERATIONAL MODES

There are three operational modes for the turn movements at an intersection. The names used to describe these modes refer to the way the turn movement is served by the controller. The three modes are as follows:

- Permitted,
- Protected, and
- Protected-permitted.

The *permitted mode* requires turning drivers to yield to conflicting traffic streams before completing the turn. Permitted left-turning drivers yield to oncoming vehicles and conflicting pedestrians. Permitted right-turning drivers yield to pedestrians. The efficiency of this mode depends on the availability of gaps in the conflicting streams. An exclusive turn lane may be provided, but it is not required. The permitted turn movement is typically presented with a circular green indication (although some agencies use other indications, such as a flashing yellow arrow). The right-turn movements in Exhibit 31-2 are operating in the permitted mode.

The *protected mode* allows turning drivers to travel through the intersection while all conflicting movements are required to yield. This mode provides for efficient turn-movement service; however, the additional turn phase typically results in increased delay to the other movements. An exclusive turn lane is typically provided with this mode. The turn phase is indicated by a green arrow signal indication. Left-turn Movements 3 and 7 in Exhibit 31-2 are operating in the protected mode.

The *protected-permitted mode* represents a combination of the permitted and protected modes. Turning drivers have the right-of-way during the associated left-turn phase. They can also complete the turn "permissively" when the adjacent through movement receives its circular green (or flashing yellow arrow) indication. This mode provides for efficient turn-movement service, often without causing a significant increase in the delay to other movements. Left-turn Movements 1 and 5 in Exhibit 31-2 are operating in the protected-permitted mode.

In general, the operational mode used for one left-turn movement is also used for the opposing left-turn movement. For example, if one left-turn movement is permitted, then so is the opposing left-turn movement. However, this agreement is not required.

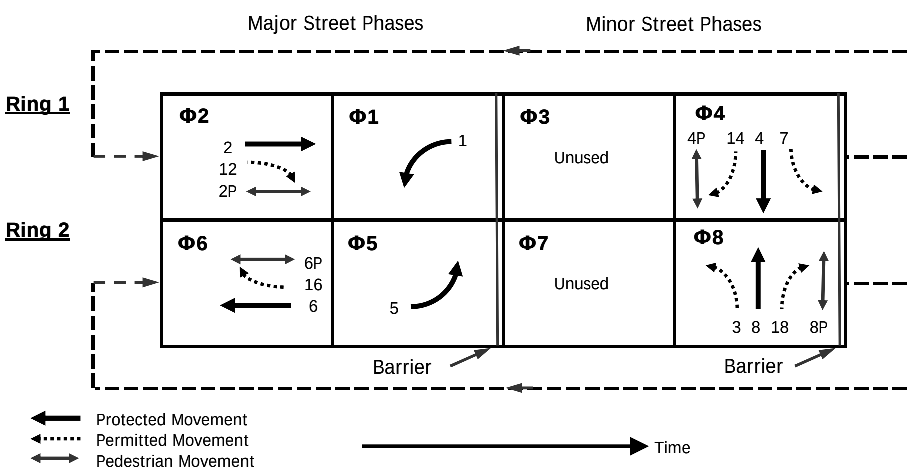
LEFT-TURN PHASE SEQUENCE

This subsection describes the sequence of service provided to left-turn movements, relative to the other intersection movements. The typical options include the following:

- No left-turn phase (i.e., permitted only),
- Leading left-turn phase,
- Lagging left-turn phase, or
- Split phasing.

The permitted-only option is used when the left-turn movement operates in the permitted mode. A left-turn phase is not provided with this option. An illustrative implementation of permitted-only phasing for left- and right-turning traffic is shown in Exhibit 31-3 for the minor street.

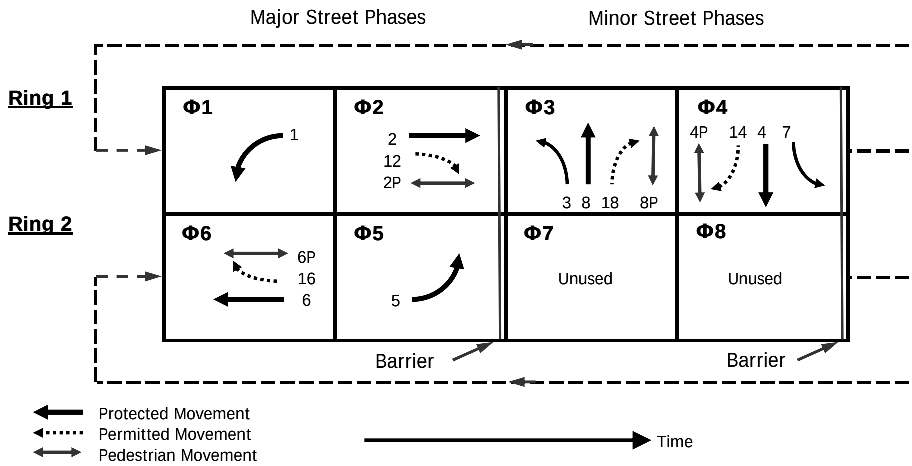
Exhibit 31-3
Illustrative Protected Lag–Lag and Permitted-Only Phasing



A leading, lagging, or split phase sequence is used when the left turn operates in the protected mode or the protected-permitted mode. The terms "leading" and "lagging" indicate the order in which the left-turn phase is presented, relative to the conflicting through movement. The leading left-turn sequence is shown in Exhibit 31-2 for the left-turn movements on the major and minor streets. The lagging left-turn sequence is shown in Exhibit 31-3 for the left-turn movements on the major street. A mix of leading and lagging phasing (called lead–lag) is shown in Exhibit 31-4 for the left-turn movements on the major street.

Split phasing describes a phase sequence in which one phase serves all movements on one approach and a second phase serves all movements on the opposing approach. Split phasing requires that all approach movements simultaneously receive a green indication. Split phasing is shown in Exhibit 31-4 for the minor street. Other variations of split phasing exist and depend on the treatment of the pedestrian movements. The left-turn movement in a split phase typically operates in the protected mode (as shown), provided that there are no conflicting pedestrian movements.

Exhibit 31-4
Illustrative Protected Lead–Lag and Split Phasing



TRAFFIC FLOW CHARACTERISTICS

This subsection describes several fundamental attributes of flow at signalized intersections. Exhibit 31-5 provides a reference for much of the discussion. The diagram represents a simple situation of vehicles on one approach to a signalized intersection during one signal cycle. The red clearance interval is not used with the phase serving the subject movement.

Exhibit 31-5 is divided into three parts. The top part shows the time–space trajectories of each vehicle on the approach as it travels "up" the page. The horizontal bar being crossed by the vehicles represents the location of the stop line. The bar's shading variations indicate the signal indication during the cycle. The middle part labels the timing intervals of interest with the symbols used throughout this chapter and Chapter 18, Signalized Intersections. The bottom part is an idealized representation of flow rate (measured at the stop line) as a function of time.

The trajectories in Exhibit 31-5 are a simplified representation of the actual vehicle position as a function of time. The trajectories shown imply that arrival and departure headways are fairly uniform during the cycle. Chapter 7, Interpreting HCM and Alternative Tool Results, provides additional discussion on the topic of vehicle trajectories, without restriction on the uniformity of headways. It also describes a procedure for using trajectory analysis to determine performance measures.

The automobile methodology described in Chapter 18 disaggregates the signal cycle into an effective green time and an effective red time for each phase to facilitate the evaluation of intersection operation. Effective green time is the time that can be used by vehicles to proceed effectively at the saturation flow rate. Effective red time for a phase is equal to the cycle length minus the effective green time. Further definitions of these variables and other basic terms are provided in Exhibit 31-6.

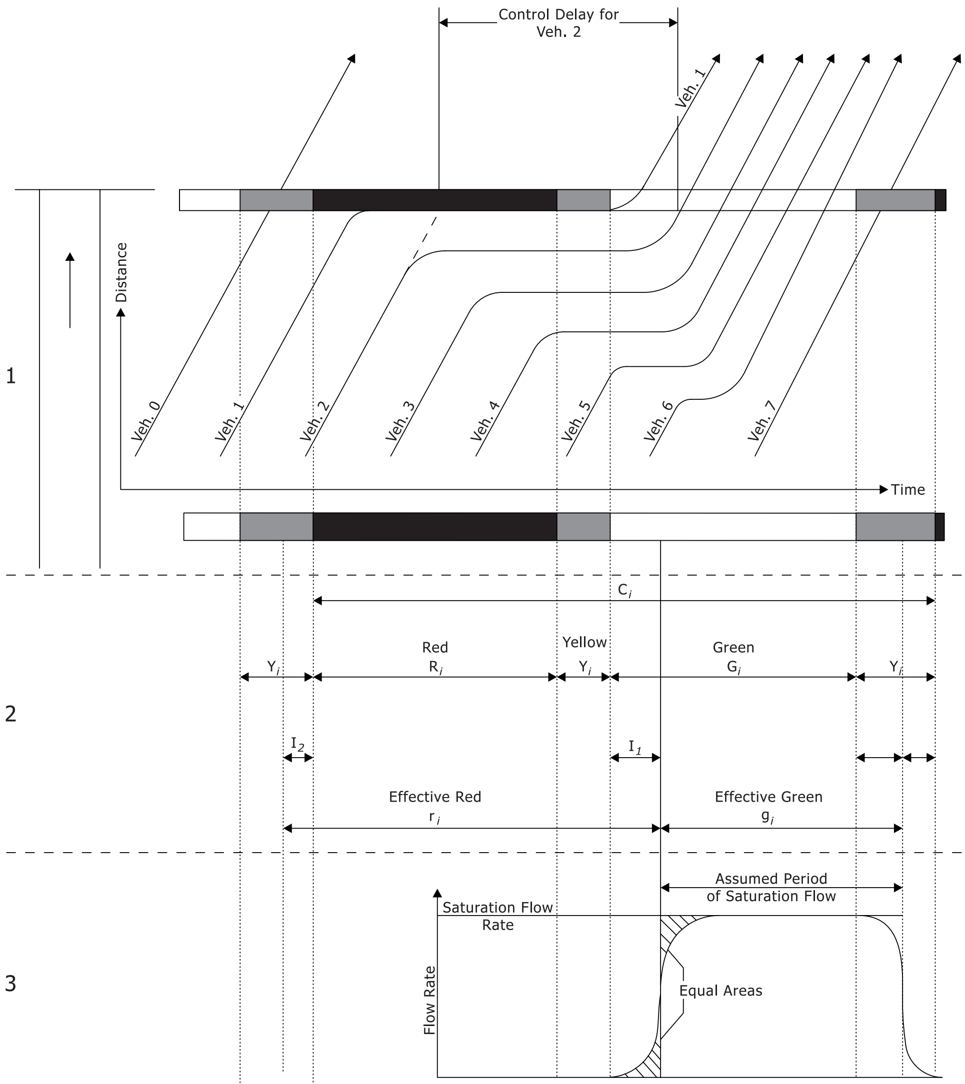


Exhibit 31-5
Fundamental Attributes of Traffic Flow at Signalized Intersections

Lost Time

Two increments of lost time are associated with a phase. At the beginning of the phase, the first few vehicles in the queue depart at headways that exceed the saturation headway. The longer headway reflects the additional time the first few drivers require to respond to the change in signal indication and accelerate to the running speed. The start-up losses are called start-up lost time l_1.

At the end of the phase, driver compliance with the yellow indication results in the latter portion of the change period (i.e., the yellow change interval and red clearance interval) not being available for vehicular service. The initial portion of the change period that is consistently used by drivers is referred to as the extension of effective green e. The remainder of the change period is considered to be clearance lost time l_2. Phase lost time l_t equals the sum of the start-up and clearance lost times.

Exhibit 31-6
Fundamental Variables of Traffic Flow at Signalized Intersections

Variable Name	Symbol	Definition
Control delay (s/veh)	d	The component of delay that results when a traffic control device causes a traffic movement to reduce speed or to stop. It represents the increase in travel time relative to the uncontrolled condition.
Clearance lost time (s)	l_2	The time at the end of a phase during which the associated traffic movements can no longer proceed effectively at the saturation flow rate.
Cycle		The time to complete one sequence of signal indications.
Cycle length (s)	C	The total time for a signal to complete one cycle.
Effective green time (s)	g	The time during which a combination of traffic movements is considered to proceed effectively at the saturation flow rate.
Effective red time (s)	r	The time during which a combination of traffic movements is not considered to proceed effectively at the saturation flow rate. It is equal to the cycle length minus the effective green time.
Extension of effective green (s)	e	The initial portion of the yellow change interval during which a combination of traffic movements is considered to proceed effectively at the saturation flow rate.
Green interval duration (s)	G	The duration of the green interval associated with a phase.
Interval		A period of time during which all signal indications remain constant.
Phase lost time (s)	l_t	The sum of the clearance lost time and start-up lost time.
Phase		The green, yellow change, and red clearance intervals assigned to a specified movement (or movements).
Red clearance interval (s)	R_c	This interval follows the yellow change interval and is optionally used to provide additional time before conflicting movements receive a green indication.
Red time (s)	R	The time in the signal cycle during which the signal indication is red for a given phase.
Adjusted saturation flow rate (veh/h/ln)	s	The equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced.
Start-up lost time (s)	l_1	The additional time consumed by the first few vehicles in a queue whose headway exceeds the saturation headway because of the need to react to the initiation of the green interval and accelerate.
Cycle lost time (s)	L	The time lost during the cycle. It represents the sum of the lost time for each critical phase.
Yellow change interval (s)	Y	This interval follows the green interval. It is used to warn drivers of the impending red indication.

The relationship between phase lost time and signal timing is shown in Equation 31-1. Research (2) has shown that start-up lost time is about 2 s and the extension of effective green is about 2 s (longer values may be appropriate for congested conditions or higher speeds). If start-up lost time equals the extension of effective green, then phase lost time is equal to the change period (i.e., $l_t = Y + R_c$).

Equation 31-1

$$l_t = l_1 + l_2 = l_1 + Y + R_c - e$$

where

l_t = phase lost time (s),

l_1 = start-up lost time = 2.0 (s),

l_2 = clearance lost time = $Y + R_c - e$ (s),

e = extension of effective green = 2.0 (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

Saturation Flow Rate

Saturation flow rate is a basic parameter used to derive capacity. It is defined in Exhibit 31-6. Saturation flow rate is expressed as an hourly rate, with units of vehicles per hour per lane (veh/h/ln).

A saturation flow rate for prevailing conditions can be determined directly from field measurement. A technique for measuring this rate is described in Section 6.

A procedure for estimating the adjusted saturation flow rate for a lane group is provided in the automobile methodology described in Chapter 18. The procedure consists of a base saturation flow rate and a series of adjustment factors. The factors are used to adjust the base rate to reflect the geometric, traffic, and environmental conditions that may influence the departure headway of vehicles in the subject lane group.

Capacity

The automobile methodology is based on the calculation of lane group capacity and its relationship to demand flow rate. Capacity is computed as the product of adjusted saturation flow rate and effective-green-to-cycle-length ratio. Capacity is defined as the maximum number of vehicles that can reasonably be expected to pass through the intersection under prevailing traffic, roadway, and signalization conditions during a 15-min period. Capacity is expressed as an hourly rate, with units of vehicles per hour.

2. CAPACITY AND PHASE DURATION

This section describes four procedures related to the calculation of capacity and phase duration. The first procedure is used to calculate the average duration of an actuated phase. The second procedure is used to calculate the lane volume distribution on multilane intersection approaches. The third procedure focuses on the calculation of phase duration for pretimed intersection operation. The fourth procedure is used to compute the pedestrian and bicycle saturation flow rate adjustment factors. Each procedure is described in a separate subsection.

ACTUATED PHASE DURATION

This subsection describes a procedure for estimating the average phase duration for an intersection that is operating with actuated control. Where appropriate, the description is extended to include techniques for estimating the duration of noncoordinated and coordinated phases. Unless stated otherwise, a noncoordinated phase is modeled as an actuated phase in this methodology.

This subsection consists of the following seven parts:

- Concepts,
- Volume computations,
- Queue accumulation polygon,
- Maximum allowable headway,
- Equivalent maximum green,
- Average phase duration, and
- Probability of max out.

The last six parts in this list describe a series of calculations that are completed in the sequence shown to obtain estimates of average phase duration and the probability of phase termination by extension to its maximum green limit (i.e., max out).

Concepts

The duration of an actuated phase is composed of five time periods, as shown in Equation 31-2. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the effective green time associated with queue clearance. The third period represents the time the green indication is extended by randomly arriving vehicles. It ends when there is a gap in traffic (i.e., gap out) or a max out. The fourth period represents the yellow change interval, and the last period represents the red clearance interval.

Equation 31-2

$$D_p = l_1 + g_s + g_e + Y + R_c$$

where

D_p = phase duration (s),

g_s = queue service time (s), and

g_e = green extension time (s).

Other variables are as previously defined.

The relationship between the variables in Equation 31-2 is shown in Exhibit 31-7 with a queue accumulation polygon. Key variables shown in the exhibit are defined in the following list:

q_r = arrival flow rate during the effective red time = $(1 - P)qC/r$ (veh/s),

P = proportion of vehicles arriving during the green indication (decimal),

r = effective red time = $C - g$ (s),

g = effective green time (s),

q_g = arrival flow rate during the effective green time = PqC/g (veh/s),

q = arrival flow rate (veh/s), and

Q_r = queue size at the end of the effective red time = $q_r r$ (veh).

Exhibit 31-7
Time Elements Influencing Actuated Phase Duration

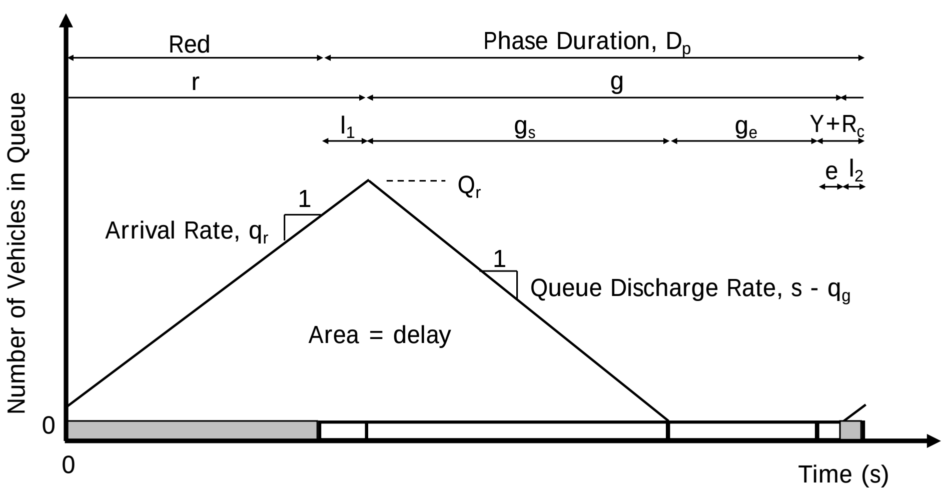


Exhibit 31-7 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the green interval starts. At this time, the queue begins to discharge at a rate equal to the saturation flow rate s less the arrival rate during green q_g. The queue clears g_s seconds after it first begins to discharge. Thereafter, random vehicle arrivals are detected and cause the green interval to be extended. Eventually, a gap occurs in traffic (or the maximum green limit is reached) and the green interval ends. The end of the green interval coincides with the end of the extension time g_e.

The effective green time for the phase is computed with the following equation:

Equation 31-3

$$g = D_p - l_1 - l_2 = g_s + g_e + e$$

Coordinated Phase Duration

The duration of a coordinated phase is dictated by the cycle length and the force-off settings for the noncoordinated phases. These settings define the points in the signal cycle where each noncoordinated phase must end. The force-off settings are used to ensure that the coordinated phases receive a green indication at a specific time in the cycle. Presumably, this time is synchronized with the coordinated phase time at the adjacent intersections so that traffic progresses along the street segment. In general, the duration of a coordinated phase is equal to the cycle length less the time allocated to the conflicting phase in the same ring and less the time allocated to the minor-street phases. Detectors are not typically assigned to the coordinated phase, and this phase is not typically extended by the vehicles it serves.

Noncoordinated Phase Duration

The duration of a noncoordinated phase is dictated by traffic demand in much the same manner as is an actuated phase. However, the noncoordinated phase duration is typically constrained by its force-off setting (rather than a maximum green setting). A noncoordinated phase is referred to here and modeled as an "actuated" phase.

Volume Computations

This part describes the calculations needed to quantify the time rate of calls submitted to the controller by the detectors. Two call rates are computed for each signal phase. The first rate represents the flow rate of calls for green extension that arrive during the green interval. The second call rate represents the flow rate of calls for phase activation that arrive during the red indication.

A. Call Rate to Extend Green

The call rate to extend the green indication for a given phase is based on the flow rate of the lane groups served by the phase. If the subject phase ends at a barrier and simultaneous gap out is enabled, then the phase's call rate is based on the lane groups it serves *plus* those groups served by the phase in the other ring that also ends at the barrier. The call rate is represented in the analysis by the flow rate parameter. This parameter represents an adjusted flow rate that accounts for the natural tendency for vehicles to form "bunches" (i.e., randomly formed platoons). The flow rate parameter for the phase is computed as follows:

Equation 31-4

$$\lambda^* = \sum_{i=1}^{m} \lambda_i$$

with

Equation 31-5

$$\lambda_i = \frac{\varphi_i q_i}{1 - \Delta_i q_i}$$

Equation 31-6

$$\varphi_i = e^{-b_i \Delta_i q_i}$$

where

λ^* = flow rate parameter for the phase (veh/s);

λ_i = flow rate parameter for lane group i (i = 1, 2, ..., m) (veh/s);

φ_i = proportion of free (unbunched) vehicles in lane group i (decimal);

q_i = arrival flow rate for lane group $i = v_i/3{,}600$ (veh/s);

v_i = demand flow rate for lane group i (veh/h);

Δ_i = headway of bunched vehicle stream in lane group i; = 1.5 s for single-lane lane group, 0.5 s otherwise (s/veh);

m = number of lane groups served during the phase; and

b_i = bunching factor for lane group i (0.6, 0.5, and 0.8 for lane groups with 1, 2, and 3 or more lanes, respectively).

It is also useful to compute the following three variables for each phase. These variables are used in a later step to compute green extension time.

Equation 31-7

$$\varphi^* = e^{-\sum_{i=1}^{m} b_i \Delta_i q_i}$$

Equation 31-8

$$\Delta^* = \frac{\sum_{i=1}^{m} \lambda_i \Delta_i}{\lambda^*}$$

Equation 31-9

$$q^* = \sum_{i=1}^{m} q_i$$

where

φ^* = combined proportion of free (unbunched) vehicles for the phase (decimal),

Δ^* = equivalent headway of bunched vehicle stream served by the phase (s/veh), and

q^* = arrival flow rate for the phase (veh/s).

The call rate for green extension for a phase that does not end at a barrier is equal to the flow rate parameter λ^*. If two phases terminate at the barrier and simultaneous gap out is enabled, then the lane group parameters for each phase are combined to estimate the call rate for green extension. Specifically, the variable m in the preceding six equations is modified to represent the combined number of lane groups served by both phases.

The following rules are evaluated to determine the number of lane groups served m if simultaneous gap out is enabled. They are described for the case in which Phases 2, 6, 4, and 8 end at the barrier (as shown in Exhibit 31-2). The rules should be modified if other phase pairs end at the barrier:

1. If Phases 2 and 6 have simultaneous gap out enabled, then the lane groups associated with Phase 2 are combined with those associated with Phase 6 in evaluating Equation 31-4 to Equation 31-9 for Phase 6.

Similarly, the lane groups associated with Phase 6 are combined with those associated with Phase 2 in evaluating these equations for Phase 2.

2. If Phases 4 and 8 have simultaneous gap out enabled, then the lane groups associated with Phase 4 are combined with those associated with Phase 8 in evaluating Phase 8. Similarly, the lane groups associated with Phase 8 are combined with those associated with Phase 4 in evaluating Phase 4.

B. Call Rate to Activate a Phase

The call rate to activate a phase is used to determine the probability that the phase is activated in the forthcoming cycle sequence. This rate is based on the arrival flow rate of the traffic movements served by the phase and whether the phase is associated with dual entry. Vehicles or pedestrians can call a phase, so a separate call rate is computed for each traffic movement.

i. Determine Phase Vehicular Flow Rate. The vehicular flow rate associated with a phase depends on the type of movements it serves as well as the approach lane allocation. The following rules apply in determining the phase vehicular flow rate:

1. If the phase exclusively serves a left-turn movement, then the phase vehicular flow rate is equal to the left-turn movement flow rate.
2. If the phase serves a through or right-turn movement and there is no exclusive left-turn phase for the adjacent left-turn movement, then the phase vehicular flow rate equals the approach flow rate.
3. If the phase serves a through or right-turn movement and there is an exclusive left-turn phase for the adjacent left-turn movement then:
    a. If there is a left-turn bay, then the phase vehicular flow rate equals the sum of the through and right-turn movement flow rates.
    b. If there is no left-turn bay, then the phase vehicular flow rate equals the approach flow rate.
    c. If split phasing is used, then the phase vehicular flow rate equals the approach flow rate.

ii. Determine Activating Vehicular Call Rate. The activating vehicular call rate q_v* is equal to the phase vehicular flow rate divided by 3,600 to convert it to units of vehicles per second. If dual entry is activated for a phase, then the activation call rate must be modified by adding its original rate to that of both concurrent phases. For example, if Phase 2 is set for dual entry, then the modified Phase 2 activation call rate equals the original Phase 2 activation call rate plus the activation rate of Phase 5 and the activation rate of Phase 6. In this manner, Phase 2 is activated when demand is present for Phase 2, 5, or 6.

iii. Determine Activating Pedestrian Call Rate. The activating pedestrian call rate q_p* is equal to the pedestrian flow rate associated with the subject approach divided by 3,600 to convert it to units of pedestrians per second. If dual entry is activated for a phase, then the activation call rate must be modified by adding its original rate to that of the opposing through phase. For example, if Phase 2 is set for dual entry, then the modified Phase 2 activation call rate equals the original

Phase 2 activation call rate plus the activation rate of Phase 6. In this manner, Phase 2 is activated when pedestrian demand is present for Phase 2 or 6.

Queue Accumulation Polygon

This part summarizes the procedure used to construct the queue accumulation polygon associated with a lane group. This polygon defines the queue size for a traffic movement as a function of time during the cycle. It is discussed at this point in Section 2 to illustrate its use in calculating queue service time. The procedure is described more fully in Section 3.

For polygon construction, all flow rate variables are converted to common units of vehicles per second per lane. The presentation in this part is based on these units for q and s. If the flow rate q exceeds the lane capacity, then it is set to equal this capacity.

A polygon is shown in Exhibit 31-7 for a through movement in an exclusive lane. At the start of the effective red, vehicles arrive at a rate of q_r and accumulate to a length of Q_r vehicles at the time the effective green begins. Thereafter, the queue begins to discharge at a rate of $s - q_g$ until it clears after g_s seconds. The queue service time g_s represents the time required to serve the queue present at the end of effective red Q_r plus any additional arrivals that join the queue before it fully clears. Queue service time is computed as $Q_r/[s - q_g]$. Substituting the variable relationships in the previous variable list into this equation yields the following equation for estimating queue service time:

$$g_s = \frac{qC(1-P)}{s/3{,}600 - qC(P/g)}$$

Equation 31-10

The polygon in Exhibit 31-7 applies to some types of lane group. Other polygon shapes are possible. A detailed procedure for constructing polygons is described in Section 3.

Maximum Allowable Headway

This part describes a procedure for calculating the maximum allowable headway (*MAH*) for the detection associated with a phase. It consists of two steps. Step A computes the *MAH* for each lane group served by the subject phase. Step B combines the *MAH* into an equivalent *MAH* for the phase. The latter step is used when a phase serves two or more lane groups or when simultaneous gap out is enabled.

The procedure addresses the situation in which there is one zone of detection per lane. This type of detection is referred to here as “stop-line detection” because the detection zone is typically located at the stop line. However, some agencies prefer to locate the detection zone at a specified distance upstream from the stop line. This procedure can be used to evaluate any single-detector-per-lane design, provided that the detector is located so that only the subject traffic movement travels over this detector during normal operation.

The detector length and detection mode input data are specified by movement group. When these data describe a through movement group, it is reasonable to assume that they also describe the detection in any shared-lane

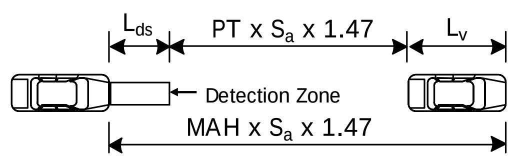
lane groups that serve the through movement. This assumption allows the movement group inputs to describe the associated lane group values, and the analysis can proceed on a lane-group basis. However, if this assumption is not valid or if information about the detection design for each lane is known, then the procedure can be extended to the calculation of *MAH* for each lane. The lane-specific *MAH*s would then be combined for the phase that serves these lanes.

Concepts

The *MAH* represents the maximum time that can elapse between successive calls for service without terminating the phase by gap out. It is useful for describing the detection design and signal settings associated with a phase. The *MAH* depends on the number of detectors serving the lane group, the length of these detectors, and the average vehicle speed in the lane group.

The relationship between passage time *PT*, detection zone length L_{ds}, vehicle length L_v, average speed S_a, and *MAH* is shown in Exhibit 31-8. The two vehicles shown are traveling from left to right and have a headway equal to the *MAH* so that the second vehicle arrives at the detector the instant the passage time is set to time out.

Exhibit 31-8
Detection Design and Maximum Allowable Headway



According to Exhibit 31-8, Equation 31-11 can be derived for estimating the *MAH* for stop-line detection operating in the presence mode.

Equation 31-11

$$MAH = PT + \frac{L_{ds} + L_v}{1.47\ S_a}$$

with

Equation 31-12

$$L_v = L_{pc}(1 - 0.01\ P_{HV}) + 0.01\ L_{HV}\, P_{HV} - D_{sv}$$

where

MAH = maximum allowable headway (s/veh),

PT = passage time setting (s),

L_{ds} = length of the stop-line detection zone (ft),

L_v = detected length of the vehicle (ft),

S_a = average speed on the intersection approach (mi/h),

L_{pc} = stored passenger car lane length = 25 (ft),

P_{HV} = percent heavy vehicles in the corresponding movement group (%),

L_{HV} = stored heavy-vehicle lane length = 45 (ft), and

D_{sv} = distance between stored vehicles = 8 (ft).

The average speed on the intersection approach can be estimated with Equation 31-13.

$$S_a = 0.90\,(25.6 + 0.47\,S_{pl})$$

Equation 31-13

where S_{pl} is the posted speed limit (mi/h).

Equation 31-11 is derived for the typical case in which the detection unit is operating in the presence mode. If it is operating in the pulse mode, then the *MAH* equals the passage time setting *PT*.

A. Determine Maximum Allowable Headway

Equation 31-11 has been modified to adapt it to various combinations of lane use and left-turn operation. A family of equations is presented in this step. The appropriate equation is selected for the subject lane group and then used to compute the corresponding *MAH*.

The equations presented in this step are derived for the typical case in which the detection unit is operating in the presence mode. If a detector is operating in the pulse mode, then the *MAH* equals the passage time setting *PT*.

The *MAH* for lane groups serving through vehicles is calculated with Equation 31-14.

$$MAH_{th} = PT_{th} + \frac{L_{ds,th} + L_v}{1.47\,S_a}$$

Equation 31-14

where

MAH_{th} = maximum allowable headway for through vehicles (s/veh),

PT_{th} = passage time setting for phase serving through vehicles (s), and

$L_{ds,th}$ = length of the stop-line detection zone in the through lanes (ft).

The *MAH* for a left-turn movement served in exclusive lanes with the protected mode (or protected-permitted mode) is based on Equation 31-14, but it is adjusted to account for the slower speed of the left-turn movement. The adjusted equation is shown as Equation 31-15.

$$MAH_{lt,e,p} = PT_{lt} + \frac{L_{ds,lt} + L_v}{1.47\,S_a} + \frac{E_L - 1}{s_o / 3{,}600}$$

Equation 31-15

where

$MAH_{lt,e,p}$ = maximum allowable headway for protected left-turning vehicles in exclusive lane (s/veh),

PT_{lt} = passage time setting for phase serving the left-turning vehicles (s),

$L_{ds,lt}$ = length of the stop-line detection zone in the left-turn lanes (ft),

E_L = equivalent number of through cars for a protected left-turning vehicle = 1.05, and

s_o = base saturation flow rate (pc/h/ln).

The *MAH* value for left-turning vehicles served in a shared lane with the protected-permitted mode is shown as Equation 31-16:

Equation 31-16

$$MAH_{lt,s,p} = MAH_{th} + \frac{E_L - 1}{s_o / 3{,}600}$$

where $MAH_{lt,s,p}$ is the maximum allowable headway for protected left-turning vehicles in a shared lane (s/veh).

The *MAH* value for left-turning vehicles served in an exclusive lane with the permitted mode is adjusted to account for the longer headway of the turning vehicle. In this case, the longer headway includes the time spent waiting for an acceptable gap in the opposing traffic stream. Equation 31-17 addresses these adjustments.

Equation 31-17

$$MAH_{lt,e} = PT_{th} + \frac{L_{ds,lt} + L_v}{1.47\ S_a} + \frac{3{,}600}{s_l} - t_{fh}$$

where

- $MAH_{lt,e}$ = maximum allowable headway for permitted left-turning vehicles in exclusive lane (s/veh);
- s_l = saturation flow rate in exclusive left-turn lane group with permitted operation (veh/h/ln); and
- t_{fh} = follow-up headway (4.5 if the subject left turn is served in a shared lane, 2.5 if the subject left turn is served in an exclusive lane) (s).

The *MAH* value for right-turning vehicles served in an exclusive lane with the protected mode is computed with Equation 31-18.

Equation 31-18

$$MAH_{rt,e,p} = PT_{rt} + \frac{L_{ds,rt} + L_v}{1.47\ S_a} + \frac{E_R - 1}{s_o / 3{,}600}$$

where

- $MAH_{rt,e,p}$ = maximum allowable headway for protected right-turning vehicles in exclusive lane (s/veh),
- PT_{rt} = passage time setting for phase serving right-turning vehicles (s),
- E_R = equivalent number of through cars for a protected right-turning vehicle = 1.18, and
- $L_{ds,rt}$ = length of the stop-line detection zone in the right-turn lanes (ft).

If the variable E_R in Equation 31-18 is divided by the pedestrian–bicycle saturation flow rate adjustment factor f_{Rpb} and PT_{th} is substituted for PT_{rt}, then the equation can be used to estimate $MAH_{rt,e}$ for permitted right-turning vehicles in an exclusive lane.

The following equations are used to estimate the *MAH* for left- and right-turning vehicles that are served in a shared lane with the permitted mode:

Equation 31-19

$$MAH_{lt,s} = MAH_{th} + \frac{3{,}600}{s_l} - t_{fh}$$

Equation 31-20

$$MAH_{rt,s} = MAH_{th} + \frac{(E_R / f_{Rpb}) - 1}{s_o / 3{,}600}$$

where $MAH_{lt,s}$ is the maximum allowable headway for permitted left-turning vehicles in a shared lane (s/veh) and $MAH_{rt,s}$ is the maximum allowable headway for permitted right-turning vehicles in a shared lane (s/veh).

B. Determine Equivalent Maximum Allowable Headway

The equivalent *MAH* (i.e., *MAH**) is calculated for cases in which more than one lane group is served by a phase. It is also calculated for phases that end at a barrier and that are specified in the controller as needing to gap out at the same time as a phase in the other ring. The following rules are used to compute the equivalent *MAH*.

1. If simultaneous gap out is not enabled, or the phase does not end at the barrier, then:
   a. If the phase serves only one movement, then the *MAH** for the phase equals the *MAH* computed for the corresponding lane group.
   b. This rule subset applies when the phase serves all movements and there is no exclusive left-turn phase for the approach (i.e., it operates with the permitted mode). The equations shown apply to the most general case in which a left-turn, through, and right-turn movement exist and a through lane group exists. If any of these movements or lane groups does not exist, then their corresponding flow rate parameter equals 0.0 veh/s.
      i. If there is no left-turn lane group or right-turn lane group (i.e., shared lanes), then the *MAH** for the phase is computed from Equation 31-21.

$$MAH^* = \frac{P_L \lambda_{sl} MAH_{lt,s} + \left[(1-P_L)\,\lambda_{sl} + \lambda_t + (1-P_R)\lambda_{sr}\right] MAH_{th} + P_R \lambda_{sr} MAH_{rt,s}}{\lambda_{sl} + \lambda_t + \lambda_{sr}}$$

Equation 31-21

where

λ_{sl} = flow rate parameter for shared left-turn and through lane group (veh/s),

λ_t = flow rate parameter for exclusive through lane group (veh/s),

λ_{sr} = flow rate parameter for shared right-turn and through lane group (veh/s),

P_L = proportion of left-turning vehicles in the shared lane (decimal), and

P_R = proportion of right-turning vehicles in the shared lane (decimal).

ii. If there is a right-turn lane group but no left-turn lane group, then Equation 31-22 is applicable.

$$MAH^* = \frac{P_L \lambda_{sl} MAH_{lt,s} + \left[(1-P_L)\,\lambda_{sl} + \lambda_t\right] MAH_{th} + \lambda_r MAH_{rt,e}}{\lambda_{sl} + \lambda_t + \lambda_r}$$

Equation 31-22

where λ_r is the flow rate parameter for the exclusive right-turn lane group (veh/s).

iii. If there is a left-turn lane group but no right-turn lane group, then the *MAH** for the phase is computed with Equation 31-23.

Equation 31-23

$$MAH^* = \frac{\lambda_l MAH_{lt,e} + \left[\lambda_t + (1 - P_R)\,\lambda_{sr}\right] MAH_{th} + P_R \lambda_{sr} MAH_{rt,s}}{\lambda_l + \lambda_t + \lambda_{sr}}$$

where λ_l is the flow rate parameter for the exclusive left-turn lane group (veh/s).

iv. If there is a left-turn lane group and a right-turn lane group, then the *MAH** for the phase is computed with Equation 31-24.

Equation 31-24

$$MAH^* = \frac{\lambda_l MAH_{lt,e} + \lambda_t MAH_{th} + \lambda_r MAH_{rt,e}}{\lambda_l + \lambda_t + \lambda_r}$$

c. If the phase serves only a through lane group, right-turn lane group, or both, then:

i. If there is a right-turn lane group and a through lane group, then the *MAH** for the phase is computed with Equation 31-25.

Equation 31-25

$$MAH^* = \frac{\lambda_t MAH_{th} + \lambda_r MAH_{rt,e}}{\lambda_t + \lambda_r}$$

ii. If there is a shared right-turn-and-through lane group, then the *MAH** for the phase is computed with Equation 31-26.

Equation 31-26

$$MAH^* = \frac{\left[\lambda_t + (1 - P_R)\,\lambda_{sr}\right] MAH_{th} + P_R \lambda_{sr} MAH_{rt,s}}{\lambda_t + \lambda_{sr}}$$

d. If the phase serves all approach movements using split phasing, then:

i. If there is one lane group (i.e., a shared lane), then the *MAH** for the phase equals the *MAH* computed for the lane group.

ii. If there is more than one lane group, then the *MAH** is computed with the equations in previous Rule 1.b but $MAH_{lt,e,p}$ is substituted for $MAH_{lt,e}$ and $MAH_{lt,s,p}$ is substituted for $MAH_{lt,s}$.

e. If the phase has protected-permitted operation with a shared left-turn and through lane, then the equations in previous Rule 1.b (i.e., 1.b.i and 1.b.ii) apply. The detection for this phasing does not influence the duration of the left-turn phase. The left-turn phase will be set to minimum recall and extend to its minimum value before terminating.

2. If simultaneous gap out is enabled and the phase ends at the barrier, then the *MAH** for the phase is computed with Equation 31-27, where the summations shown are for all lane groups served by the subject (or concurrent) phase.

Equation 31-27

$$MAH^* = \frac{MAH \sum \lambda_i + MAH_c \sum \lambda_{c,i}}{\sum \lambda_i + \sum \lambda_{c,i}}$$

where

MAH^* = equivalent maximum allowable headway for the phase (s/veh),

MAH_c = maximum allowable headway for the concurrent phase that also ends at the barrier (s/veh), and

$\lambda_{c,i}$ = flow rate parameter for lane group *i* served in the concurrent phase that also ends at the barrier (veh/s).

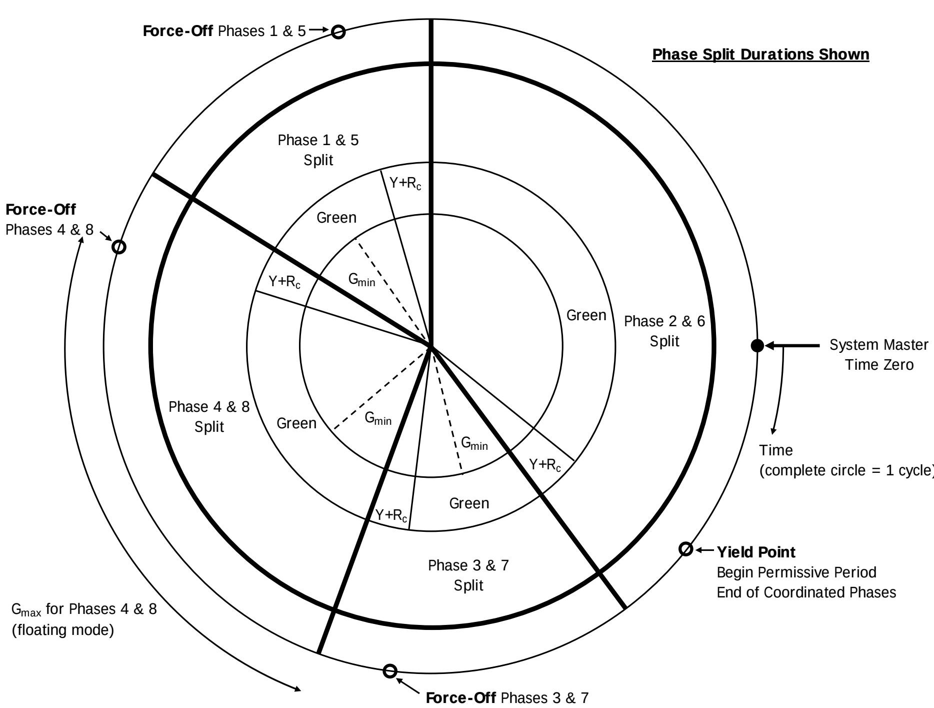
When there is split phasing, there are no concurrent phases and Equation 31-27 does not apply.

Equivalent Maximum Green

In coordinated-actuated operation, the force-off points are used to constrain the duration of the noncoordinated phases. The maximum green setting is also available to provide additional constraint; however, it is not commonly used. In fact, the default mode in most modern controllers is to inhibit the maximum green timer when the controller is used in a coordinated signal system.

The relationship between the force-off points, yield point, and phase splits is shown in Exhibit 31-9. The yield point is associated with the coordinated phases (i.e., Phases 2 and 6). It coincides with the start of the yellow change interval. If a call for service by one of the noncoordinated phases arrives after the yield point is reached, then the coordinated phases begin the termination process by presenting the yellow indication. Calls that arrive before the yield point are not served until the yield point is reached.

Exhibit 31-9
Force-Off Points, Yield Point, and Phase Splits



The force-off and yield points for common phase pairs are shown in Exhibit 31-9 to occur at the same time. This approach is shown for convenience of

illustration. In practice, the two phases may have different force-off or yield points.

A permissive period typically follows the yield point. If a conflicting call arrives during the permissive period, then the phase termination process begins immediately and all phases associated with conflicting calls are served in sequence. Permissive periods are typically long enough to ensure that all calls for service are met during the signal cycle. This methodology does not explicitly model permissive periods. It is assumed that the permissive period begins at the yield point and is sufficiently long that all conflicting calls are served in sequence each cycle.

One force-off point is associated with each of Phases 1, 3, 4, 5, 7, and 8. If a phase is extended to its force-off point, the phase begins the termination process by presenting the yellow indication (phases that terminate at a barrier must be in agreement to terminate before the yellow indication will be presented). Modern controllers compute the force-off points and yield point by using the entered phase splits and change periods based on the relationships shown in Exhibit 31-9.

The concept of “equivalent” maximum green is useful for modeling noncoordinated phase operation. This maximum green replicates the effect of a force-off or yield point on phase duration. The procedure described in this part is used to compute the equivalent maximum green for coordinated-actuated operation. Separate procedures are described for the “fixed” force mode and the “floating” force mode.

A. Determine Equivalent Maximum Green for Floating Force Mode

This step is applicable if the controller is set to operate in the floating force mode. With this mode, each noncoordinated phase has its force-off point set at the split time after the phase first becomes active. The force-off point for a phase is established when the phase is first activated. Thus, the force-off point “floats,” or changes, each time the phase is activated. This operation allows unused split time to revert to the coordinated phase via an early return to green. The equivalent maximum green for this mode is computed as being equal to the phase split less the change period. This relationship is shown in Exhibit 31-9 for Phases 4 and 8.

B. Determine Equivalent Maximum Green for Fixed Force Mode

This step is applicable if the controller is set to operate in the fixed force mode. With this mode, each noncoordinated phase has its force-off point set at a fixed time in the cycle, relative to time zero on the system master. The force-off points are established whenever a new timing plan is selected (e.g., by time of day) and remains “fixed” until a new plan is selected. This operation allows unused split time to revert to the following phase.

The equivalent maximum green for this mode is computed for each phase. It is computed by first establishing the fixed force-off points (as shown in Exhibit 31-9) and then computing the average duration of each noncoordinated phase. The calculation process is iterative. For the first calculation of average

phase duration, the maximum green is equal to the phase split less the change period. Thereafter, the maximum green for a specific phase is computed as the difference between its force-off point and the sum of the previous phases, starting with the first noncoordinated phase. Equation 31-28 illustrates this computation for Phase 4, using the ring structure shown in Exhibit 31-2. A similar calculation is performed for the other phases.

Equation 31-28

$$G_{max,4} = FO_4 - (YP_2 + CP_2 + G_3 + CP_3)$$

where

$G_{max,4}$ = equivalent maximum green for Phase 4 (s),

FO_4 = force-off point for Phase 4 (s),

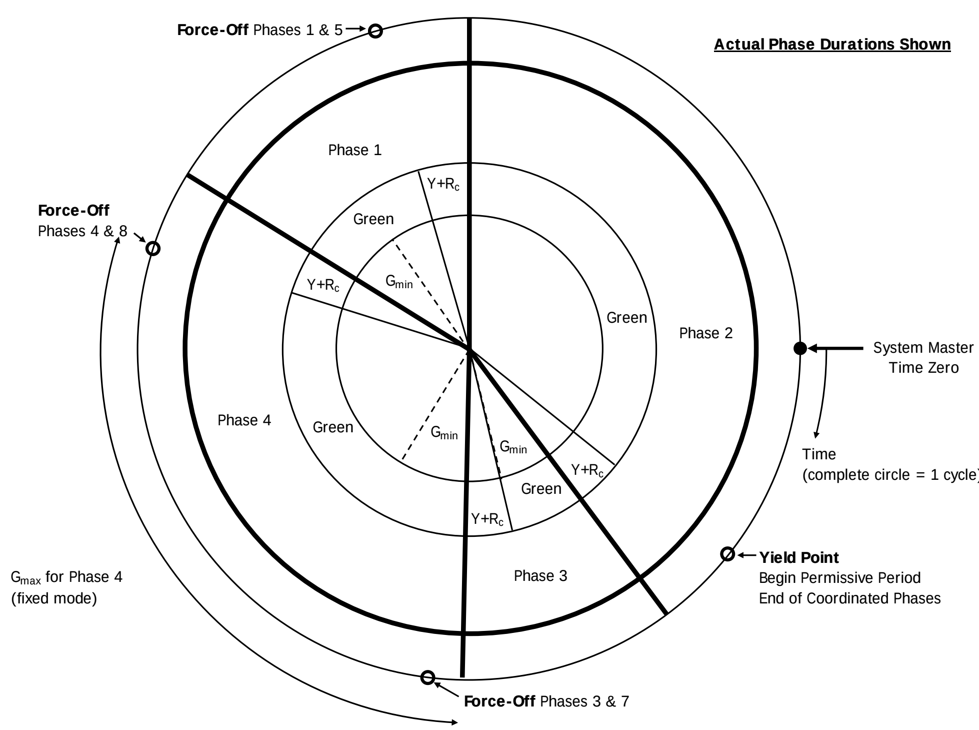
YP_2 = yield point for Phase 2 (s),

G_3 = green interval duration for Phase 3 (s), and

CP_i = change period (yellow change interval plus red clearance interval) for phase *i* (s).

The maximum green obtained from this equation is shown in Exhibit 31-10 for the ring that serves Phases 1, 2, 3, and 4. Unlike Exhibit 31-9, Exhibit 31-10 illustrates the *actual* average phase durations for a given cycle. In this example, Phase 3 timed to its minimum green and terminated. It never reached its force-off point. The unused time from Phase 3 was made available to Phase 4, which resulted in a larger maximum green than was obtained with the floating mode (see Exhibit 31-9). If every noncoordinated phase extends to its force-off point, then the maximum green from the fixed force mode equals that obtained from the floating force mode.

Exhibit 31-10
Example Equivalent Maximum Green for Fixed Force Mode



Average Phase Duration

This part describes the sequence of calculations needed to estimate the average duration of a phase. In fact, the process requires the combined calculation of the duration of all phases together because of the constraints imposed by the controller ring structure and associated barriers.

The calculation process is iterative because several intermediate equations require knowledge of the green interval duration. Specifically, the green interval duration is required in calculating lane group flow rate, queue service time, permitted green time, left-turn volume served during the permitted portion of a protected-permitted mode, and equivalent maximum green. To overcome this circular dependency, the green interval for each phase is initially estimated and then the procedure is implemented by using this estimate. When completed, the procedure provides a new initial estimate of the green interval duration. The calculations are repeated until the initial estimate and computed green interval duration are effectively equal.

The calculation steps that compose the procedure are described in the following paragraphs.

A. Compute Effective Change Period

The change period is computed for each phase. It is equal to the sum of the yellow change interval and the red clearance interval (i.e., $Y + R_c$). For phases that end at a barrier, the longer change period of the two phases that terminate at a barrier is used to define the effective change period for both phases.

B. Estimate Green Interval

An initial estimate of the green interval duration is provided for each phase. For the first iteration with fully actuated control, the initial estimate is equal to the maximum green setting. For the first iteration with coordinated-actuated control, the initial estimate is equal to the input phase split less the change period.

C. Compute Equivalent Maximum Green (Coordinated-Actuated)

If the controller is operating as coordinated-actuated, then the equivalent maximum green is computed for each phase. It is based on the estimated green interval duration, phase splits, and change periods. The previous part titled Equivalent Maximum Green described how to compute this value.

D. Construct Queue Accumulation Polygon

The queue accumulation polygon is constructed for each lane group and corresponding phase by using the known flow rates and signal timing. The procedure for constructing this polygon was summarized in the previous part titled Queue Accumulation Polygon. It is described in more detail in Section 3.

E. Compute Queue Service Time

The queue service time g_s is computed for each queue accumulation polygon constructed in the previous step. For through movements, or left-turn movements served during a left-turn phase, the polygon in Exhibit 31-7 applies

and Equation 31-10 can be used. The procedure described in Section 3 is applicable to more complicated polygon shapes.

F. Compute Call Rate to Extend Green

The extending call rate is represented as the flow rate parameter λ. This parameter is computed for each lane group served by an actuated phase and then aggregated to a phase-specific value. The procedure for computing this parameter is described in the previous part titled Volume Computations.

G. Compute Equivalent Maximum Allowable Headway

The equivalent maximum allowable headway MAH^* is computed for each actuated phase. The procedure for computing the MAH^* is described in the previous part titled Maximum Allowable Headway.

H. Compute Number of Extensions Before Max Out

The average number of extensions before the phase terminates by max out is computed for each actuated phase with the following equation:

$$n = q^* \left[G_{max} - (g_s + l_1) \right] \quad \geq 0.0$$

Equation 31-29

where n is the number of extensions before the green interval reaches its maximum limit, G_{max} is the maximum green setting (s), and other variables are as previously defined.

I. Compute Probability of Green Extension

The probability of the green interval being extended by randomly arriving vehicles is computed for each actuated phase with the following equation:

$$p = 1 - \varphi^* e^{-\lambda^* (MAH^* - \Delta^*)}$$

Equation 31-30

where p is the probability of a call headway being less than the maximum allowable headway and other variables are as previously defined.

J. Compute Green Extension Time

The average green extension time is computed for each actuated phase with the following equation:

$$g_e = \frac{p^2 (1 - p^n)}{q^* (1 - p)}$$

Equation 31-31

K. Compute Activating Call Rate

The call rate to activate a phase is computed for each actuated phase. A separate rate is computed for vehicular traffic and for pedestrian traffic. The rate for each travel mode is based on its flow rate and the use of dual entry. The procedure for computing this rate is described in the previous part titled Volume Computations.

L. Compute Probability of Phase Call

The probability that an actuated phase is called depends on whether it is set on recall in the controller. If it is on recall, then the probability that the phase is called equals 1.0. If the phase is not on recall, then the probability that it is called can be estimated with Equation 31-32 to Equation 31-34.

Equation 31-32

$$p_c = p_v(1-p_p) + p_p(1-p_v) + p_v p_p$$

with

Equation 31-33

$$p_v = 1 - e^{-q_v^* C}$$

Equation 31-34

$$p_p = 1 - e^{-q_p^* P_p C}$$

where

p_c = probability that the subject phase is called,

p_v = probability that the subject phase is called by a vehicle detection,

p_p = probability that the subject phase is called by a pedestrian detection,

q_v^* = activating vehicular call rate for the phase (veh/s),

q_p^* = activating pedestrian call rate for the phase (p/s), and

P_p = probability of a pedestrian pressing the detector button = 0.51.

Other variables are as previously defined.

The probability of a pedestrian pressing the detector button reflects the tendency of some pedestrians to decline from using the detector button before crossing a street. Research indicates that about 51% of all crossing pedestrians will push the button to place a call for pedestrian service (*3*).

M. Compute Unbalanced Green Duration

The unbalanced average green interval duration is computed for each actuated phase with Equation 31-35 to Equation 31-37.

Equation 31-35

$$G_u = G_{|veh,call}\, p_v(1-p_p) + G_{|ped,call}\, p_p(1-p_v) + \max\left[G_{|veh,call}, G_{|ped,call}\right] p_v p_p \quad \le G_{max}$$

with

Equation 31-36

$$G_{|veh,call} = \max\begin{bmatrix} l_1 + g_s + g_e \\ G_{min} \end{bmatrix}$$

Equation 31-37

$$G_{|ped,call} = Walk + PC$$

where

G_u = unbalanced green interval duration for a phase (s),

$G_{|veh,call}$ = average green interval given that the phase is called by a vehicle detection (s),

G_{min} = minimum green setting (s),

$G_{|ped,call}$ = average green interval given that the phase is called by a pedestrian detection (s),

$Walk$ = pedestrian walk setting (s), and

PC = pedestrian clear setting (s).

If maximum recall is set for the phase, then G_u is equal to G_{max}. If the phase serves a left-turn movement that operates in the protected mode, then the probability that it is called by pedestrian detection p_p is equal to 0.0.

If the phase serves a left-turn movement that operates in the protected-permitted mode and the left-turn movement shares a lane with through vehicles, then the green interval duration is equal to the phase's minimum green setting.

The green interval duration obtained from this step is "unbalanced" because it does not reflect the constraints imposed by the controller ring structure and associated barriers. These constraints are imposed in Step O or Step P, depending on the type of control used at the intersection.

It is assumed that the rest-in-walk mode is not enabled.

N. Compute Unbalanced Phase Duration

The unbalanced average phase duration is computed for each actuated phase by adding the unbalanced green interval duration and the corresponding change period components. This calculation is completed with Equation 31-38.

$$D_{up} = G_u + Y + R_c$$

Equation 31-38

where D_{up} is the unbalanced phase duration (s).

If simultaneous gap out is enabled, the phase ends at a barrier, and the subject phase experiences green extension when the concurrent phase has reached its maximum green limit, then both phases are extended but only due to the call flow rate of the subject phase. Hence, the green extension time computed in Step J is too long. The effect is accounted for in the current step by multiplying the green extension time from Step J by a "flow rate ratio." This ratio represents the sum of the flow rate parameter for each lane group served by the subject phase divided by the sum of the flow rate parameter for each group served by the subject phase and served by the concurrent phase (the latter sum equals the call rate from Step F).

O. Compute Average Phase Duration—Fully Actuated Control

For this discussion, it is assumed that Phases 2 and 6 are serving Movements 2 and 6, respectively, on the major street (see Exhibit 31-2). If the left-turn movements on the major street operate in the protected mode or the protected-permitted mode, then Movements 1 and 5 are served during Phases 1 and 5, respectively. Similarly, Phases 4 and 8 are serving Movements 4 and 8, respectively, on the minor street. If the left-turn movements on the minor street are protected or protected-permitted, then Phases 3 and 7 are serving Movements 3 and 7, respectively. If a through movement phase occurs first in a phase pair, then the other phase (i.e., the one serving the opposing left-turn movement) is a "lagging" left-turn phase.

The following rules are used to estimate the average duration of each phase:

1. Given two phases that occur in sequence between barriers (i.e., phase a followed by phase b), the duration of $D_{p,a}$ is equal to the unbalanced

phase duration of the first phase to occur (i.e., $D_{p,a} = D_{up,a}$). The duration of $D_{p,b}$ is based on Equation 31-39 for the major-street phases.

Equation 31-39

$$D_{p,b} = \max[D_{up,1} + D_{up,2}, D_{up,5} + D_{up,6}] - D_{p,a}$$

where

$D_{p,b}$ = phase duration for phase *b*, which occurs just after phase *a* (s);

$D_{p,a}$ = phase duration for phase *a*, which occurs just before phase *b* (s); and

$D_{up,i}$ = unbalanced phase duration for phase *i*; *i* = 1, 2, 5, and 6 for major street and *i* = 3, 4, 7, and 8 for minor street (s).

Equation 31-40 applies for the minor street phases:

Equation 31-40

$$D_{p,b} = \max[D_{up,3} + D_{up,4}, D_{up,7} + D_{up,8}] - D_{p,a}$$

For example, if the phase pair consists of Phase 3 followed by Phase 4 (i.e., a leading left-turn arrangement), then $D_{p,3}$ is set to equal $D_{up,3}$ and $D_{p,4}$ is computed from Equation 31-40. In contrast, if the pair consists of Phase 8 followed by Phase 7 (i.e., a lagging left-turn arrangement), then $D_{p,8}$ is set to equal $D_{up,8}$, and $D_{p,7}$ is computed from Equation 31-40.

2. If an approach is served with one phase operating in the permitted mode (but not split phasing), then $D_{p,a}$ equals 0.0 and the equations above are used to estimate the duration of the phase (i.e., $D_{p,b}$).
3. If split phasing is used, then $D_{p,a}$ equals the unbalanced phase duration for one approach and $D_{p,b}$ equals the unbalanced phase duration for the other approach.

P. Compute Average Phase Duration—Coordinated-Actuated Control

For this discussion, it is assumed that Phases 2 and 6 are the coordinated phases serving Movements 2 and 6, respectively (see Exhibit 31-2). If the left-turn movements operate in the protected mode or the protected-permitted mode, then the opposing left-turn movements are served during Phases 1 and 5. If a coordinated phase occurs first in the phase pair, then the other phase (i.e., the one serving the opposing left-turn movement) is a "lagging" left-turn phase.

The following rules are used to estimate the average duration of each phase:

1. If the phase is associated with the street serving the coordinated movements, then:
   a. If a left-turn phase exists for the subject approach, then its duration $D_{p,l}$ equals $D_{up,l}$, and the opposing through phase has a duration $D_{p,t}$ based on Equation 31-41.

Equation 31-41

$$D_{p,t} = C - \max[D_{up,3} + D_{up,4}, D_{up,7} + D_{up,8}] - D_{p,l}$$

where $D_{p,t}$ is the phase duration for coordinated phase *t* (*t* = 2 or 6) (s), $D_{p,l}$ is the phase duration for left-turn phase *l* (*l* = 1 or 5) (s), and other variables are as previously defined.

If Equation 31-41 is applied to Phase 2, then *t* equals 2 and *l* equals 1. If it is applied to Phase 6, then *t* equals 6 and *l* equals 5.

b. If a left-turn phase does not exist for the subject approach, then $D_{p,l}$ equals 0.0 and Equation 31-41 is used to estimate the duration of the coordinated phase.

This procedure for determining average phase duration accommodates split phasing only on the street that does not serve the coordinated movements.

If $D_{p,t}$ obtained from Equation 31-41 is less than the minimum phase duration (= $G_{min} + Y + R_c$), then the phase splits are too generous and do not leave adequate time for the coordinated phases.

2. If the phase is associated with the street serving the noncoordinated movements, then the rules described in Step O are used to determine the phase's average duration.

Q. Compute Green Interval Duration

The average green interval duration is computed for each phase by subtracting the yellow change and red clearance intervals from the average phase duration, as shown in Equation 31-42.

$$G = D_p - Y - R_c$$

Equation 31-42

where G is the green interval duration (s), and other variables are as previously defined.

R. Compare Computed and Estimated Green Interval Durations

The green interval duration from the previous step is compared with the value estimated in Step B. If the two values differ by 0.1 s or more, then the computed green interval becomes the "new" initial estimate and the sequence of calculations is repeated starting with Step C. This process is repeated until the two green intervals differ by less than 0.1 s.

If the intersection is semiactuated or fully actuated, the equilibrium cycle length is computed with Equation 31-43:

$$C_e = \sum_{i=1}^{4} D_{p,i}$$

Equation 31-43

where C_e is the equilibrium cycle length (s), i is the phase number, and other variables are as previously defined. The sum in this equation includes all phases in Ring 1. The equilibrium cycle length is used in all subsequent calculations where cycle length C is an input variable.

Probability of Max Out

When the green indication is extended to its maximum green limit, the associated phase is considered to have terminated by max out. The probability of max out can be equated to the joint probability of there being a sequence of calls to the phase in service, each call having a headway that is shorter than the equivalent maximum allowable headway for the phase. This probability can be stated mathematically with Equation 31-44.

Equation 31-44

$$p_x = p^{n_x}$$

with

Equation 31-45

$$n_x = \frac{G_{max} - MAH^* - (g_s + l_1)}{h} \quad \geq 0.0$$

Equation 31-46

$$h = \frac{\Delta^* + \varphi^* / \lambda^* - (MAH^* + 1/\lambda^*)\, \varphi^* e^{-\lambda^* (MAH^* - \Delta^*)}}{1 - \varphi^* e^{-\lambda^* (MAH^* - \Delta^*)}}$$

where

p_x = probability of phase termination by extension to the maximum green limit,

n_x = number of calls necessary to extend the green to max out, and

h = average call headway for all calls with headways less than MAH^* (s).

Other variables are as previously defined.

LANE GROUP FLOW RATE ON MULTIPLE-LANE APPROACHES

Introduction

When drivers approach an intersection, their primary criterion for lane choice is movement accommodation (i.e., left, through, or right). If multiple exclusive lanes are available to accommodate their movement, they tend to choose the lane that minimizes their service time (i.e., the time required to reach the stop line, as influenced by the number and type of vehicles between them and the stop line). This criterion tends to result in relatively equal lane use under most circumstances.

If one of the lanes being considered is a shared lane, then service time is influenced by the distribution of turning vehicles in the shared lane. Turning vehicles tend to have a longer service time because of the turn maneuver. Moreover, when turning vehicles operate in the permitted mode, their service time can be lengthy because of the gap search process.

Observation of driver lane-choice behavior indicates that there is an equilibrium lane flow rate that characterizes the collective choices of the population of drivers. Research indicates that the equilibrium flow rate can be estimated from the lane volume distribution that yields the minimum service time for the population of drivers having a choice of lanes (*4*).

A model for predicting the equilibrium lane flow rate on an intersection approach is described in this subsection. The model is based on the principle that through drivers will choose the lane that minimizes their perceived service time. As a result of this lane selection process, each lane will have the same minimum service time. The principle is represented mathematically by (*a*) defining service time for each lane as the product of lane flow rate and saturation headway, (*b*) representing this product as the lane demand-to-saturation-flow-rate ratio (i.e.,

v/s ratio), and (c) making the v/s ratios equal among alternative approach lanes. Equation 31-47 is derived from this representation.

Equation 31-47

$$\frac{v_i}{s_i} = \frac{\sum_{i=1}^{N_{th}} v_i}{\sum_{i=1}^{N_{th}} s_i}$$

where

v_i = demand flow rate in lane i (veh/h/ln),

s_i = saturation flow rate in lane i (veh/h/ln), and

N_{th} = number of through lanes (shared or exclusive) (ln).

The "equalization of flow ratios" principle has been embodied in the HCM since the 1985 edition. Specifically, it has been used to derive the equation for estimating the proportion of left-turning vehicles in a shared lane P_L.

During field observations of various intersection approaches, it was noted that the principle overestimated the effect of turning vehicles in shared lanes for very low and for very high approach flow rate conditions (*5*). Under low flow rate conditions, it was rationalized that through drivers are not motivated to change lanes because the frequency of turns is very low and the threat of delay is negligible. Under high-flow-rate conditions, it was rationalized that through drivers do not have an opportunity to change lanes because of the lack of adequate gaps in the outside lane. The field observations also indicated that most lane choice decisions (and related lane changes) for through drivers tended to occur upstream of the intersection, before deceleration occurs.

As a result of the aforementioned field observations, the model was extended to include the probability of a lane change. The probability of a lane change represents the joint probability of there being motivation (i.e., moderate to high flow rates) and opportunity (i.e., adequate lane change gaps). A variable that is common to each probability distribution is the ratio of the approach flow rate to the maximum flow rate that would allow any lane changes. This maximum flow rate is the rate corresponding to the minimum headway considered acceptable for a lane change (i.e., about 3.7 s) (*6*). Exhibit 31-11 illustrates the modeled relationship between lane change probability and the flow ratio in the traffic lanes upstream of the intersection, before deceleration occurs (*5*).

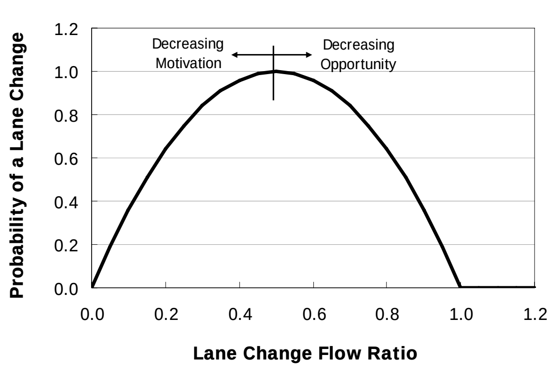


Exhibit 31-11
Probability of a Lane Change

LIVE GRAPH
Click here to view

Procedure

The procedure described in this part is generalized so that it can be applied to any signalized intersection approach with any combination of exclusive turn lanes, shared lanes, and exclusive through lanes. At least one shared lane must be present, and the approach must have two or more lanes (or bays) serving two or more traffic movements. This type of generalized formulation is attractive because of its flexibility; however, the trade-off is that the calculation process is iterative. If a closed-form solution is desired, then one would likely have to be uniquely derived for each lane assignment combination.

The procedure is described in the following steps. Input variables used in the procedure are identified in the following list and are shown in Exhibit 31-12:

N_l = number of lanes in exclusive left-turn lane group (ln),

N_{sl} = number of lanes in shared left-turn and through lane group (ln),

N_t = number of lanes in exclusive through lane group (ln),

N_{sr} = number of lanes in shared right-turn and through lane group (ln),

N_r = number of lanes in exclusive right-turn lane group (ln),

N_{lr} = number of lanes in shared left- and right-turn lane group (ln),

v_{lt} = left-turn demand flow rate (veh/h),

v_{th} = through demand flow rate (veh/h),

v_{rt} = right-turn demand flow rate (veh/h),

v_l = demand flow rate in exclusive left-turn lane group (veh/h/ln),

v_{sl} = demand flow rate in shared left-turn and through lane group (veh/h),

v_t = demand flow rate in exclusive through lane group (veh/h/ln),

v_{sr} = demand flow rate in shared right-turn and through lane group (veh/h),

v_r = demand flow rate in exclusive right-turn lane group (veh/h/ln),

v_{lr} = demand flow rate in shared left- and right-turn lane group (veh/h),

$v_{sl,lt}$ = left-turn flow rate in shared lane group (veh/h/ln),

$v_{sr,rt}$ = right-turn flow rate in shared lane group (veh/h/ln),

s_l = saturation flow rate in exclusive left-turn lane group with permitted operation (veh/h/ln),

s_{sl} = saturation flow rate in shared left-turn and through lane group with permitted operation (veh/h/ln),

s_t = saturation flow rate in exclusive through lane group (veh/h/ln),

s_{sr} = saturation flow rate in shared right-turn and through lane group with permitted operation (veh/h/ln),

s_r = saturation flow rate in exclusive right-turn lane group with permitted operation (veh/h/ln),

s_{lr} = saturation flow rate in shared left- and right-turn lane group (veh/h/ln),

s_{th} = saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, and area type) (veh/h/ln),

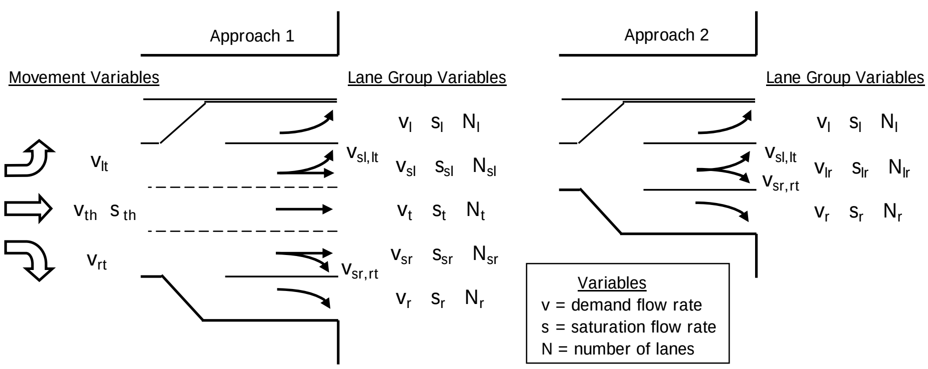
g_p = effective green time for permitted left-turn operation (s),

g_f = time before the first left-turning vehicle arrives and blocks the shared lane (s), and

g_u = duration of permitted left-turn green time that is not blocked by an opposing queue (s).

Each shared-lane lane group has one lane (i.e., $N_{sl} = 1$, $N_{sr} = 1$, and $N_{lr} = 1$). Procedures for calculating g_p, g_f, and g_u are provided in Section 3.

Exhibit 31-12
Input Variables for Lane Group Flow Rate Procedure



A. Compute Modified Through-Car Equivalents

Three modified through-car equivalent factors are computed for the left-turn movement. These factors are computed with Equation 31-48 through Equation 31-52.

$$E_{L,m} = (E_L - 1)P_{lc} + 1$$ **Equation 31-48**

$$E_{L1,m} = \left(\frac{E_{L1}}{f_{Lpb}} - 1\right)P_{lc} + 1$$ **Equation 31-49**

$$E_{L2,m} = \left(\frac{E_{L2}}{f_{Lpb}} - 1\right)P_{lc} + 1$$ **Equation 31-50**

with

$$P_{lc} = 1 - \left(\left[2\frac{v_{app}}{s_{lc}}\right] - 1\right)^2 \geq 0.0$$ **Equation 31-51**

$$v_{app} = \frac{v_{lt} + v_{th} + v_{rt}}{N_{sl} + N_t + N_{sr}}$$ **Equation 31-52**

where

$E_{L,m}$ = modified through-car equivalent for a protected left-turning vehicle,

$E_{L1,m}$ = modified through-car equivalent for a permitted left-turning vehicle,

E_{L1} = equivalent number of through cars for a permitted left-turning vehicle,

$E_{L2,m}$ = modified through-car equivalent for a permitted left-turning vehicle when opposed by a queue on a single-lane approach,

E_{L2} = equivalent number of through cars for a permitted left-turning vehicle when opposed by a queue on a single-lane approach,

f_{Lpb} = pedestrian adjustment factor for left-turn groups,

P_{lc} = probability of a lane change among the approach through lanes,

v_{app} = average demand flow rate per through lane (upstream of any turn bays on the approach) (veh/h/ln),

s_{lc} = maximum flow rate at which a lane change can occur = 3,600/t_{lc} (veh/h/ln), and

t_{lc} = critical merge headway = 3.7 (s).

The factor obtained from Equation 31-50 is applicable when permitted left-turning vehicles are opposed by a queue on a single-lane approach. Equations for calculating E_{L1} and E_{L2} are provided in Section 3. A procedure for calculating f_{Lpb} is provided later in this section.

If the approach has a shared left- and right-turn lane on the approach (as shown in Approach 2 in Exhibit 31-12), then Equation 31-53 is used to compute the average demand flow rate per lane (with N_{lr} = 1.0).

Equation 31-53

$$v_{app} = (v_{lt} + v_{rt}) / N_{lr}$$

The modified through-car equivalent for permitted right-turning vehicles is computed with Equation 31-54.

Equation 31-54

$$E_{R,m} = \left(\frac{E_R}{f_{Rpb}} - 1 \right) P_{lc} + 1$$

where $E_{R,m}$ is the modified through-car equivalent for a protected right-turning vehicle, f_{Rpb} is the pedestrian–bicycle adjustment factor for right-turn groups, and other variables are as previously defined.

A procedure for calculating f_{Rpb} is provided later in this section.

B. Estimate Shared-Lane Lane Group Flow Rate

The procedure requires an initial estimate of the demand flow rate for each traffic movement in each shared-lane lane group on the subject approach. For the shared lane serving left-turn and through vehicles, the left-turn flow rate in the shared lane $v_{sl,lt}$ is initially estimated as 0.0 veh/h and the total lane group flow rate v_{sl} is estimated as equal to the average flow rate per through lane v_{app}. For the shared lane serving right-turn vehicles, the right-turn flow rate in the shared lane $v_{sr,rt}$ is estimated as 0.0 veh/h and the total lane group flow rate v_{sr} is estimated as

equal to the average flow rate per through lane v_{app}. These estimates will be updated in a subsequent step.

C. Compute Exclusive Lane Group Flow Rate

The demand flow rate in the exclusive left-turn lane group v_l is computed with Equation 31-55, where all variables are as previously defined.

$$v_l = \frac{v_{lt} - v_{sl,lt}}{N_l} \geq 0.0$$

Equation 31-55

A similar calculation is completed to estimate the demand flow rate in the exclusive right-turn lane group v_r. Then, the flow rate in the exclusive through lane group is computed with Equation 31-56.

$$v_t = \frac{v_{th} - v_{sl,lt} - v_{sr,rt}}{N_t} \geq 0.0$$

Equation 31-56

D. Compute Proportion of Turns in Shared-Lane Lane Groups

The proportion of left-turning vehicles in the shared left-turn and through lane is computed with Equation 31-57.

$$P_L = \frac{v_{sl,lt}}{v_{sl}} \leq 1.0$$

Equation 31-57

where P_L is the proportion of left-turning vehicles in the shared lane. Substitution of $v_{sr,rt}$ for $v_{sl,lt}$ and v_{sr} for v_{sl} in Equation 31-57 yields an estimate of the proportion of right-turning vehicles in the shared lane P_R.

The proportion of left-turning vehicles in the shared left- and right-turn lane is computed with Equation 31-58.

$$P_L = \frac{v_{sl,lt}}{v_{lr}} \leq 1.0$$

Equation 31-58

Substituting $v_{sr,rt}$ for $v_{sl,lt}$ in Equation 31-58 yields an estimate of the proportion of right-turning vehicles in the shared lane P_R.

E. Compute Lane Group Saturation Flow Rate

The saturation flow rate for the lane group shared by the left-turn and through movements is computed with Equation 31-59.

$$s_{sl} = \frac{s_{th}}{g_p}\left(g_f + \frac{g_{diff}}{1 + P_L\left(E_{L2,m} - 1\right)} + \frac{\min(g_p - g_f, g_u)}{1 + P_L\left(E_{L1,m} - 1\right)}\right) \geq s_{sl,\min}$$

Equation 31-59

with

$$s_{sl,\min} = (1 + P_L)\frac{3{,}600}{g_p}$$

Equation 31-60

where g_{diff} is the supplemental service time for shared single-lane approaches (s), $s_{sl,min}$ is the minimum saturation flow rate in shared left-turn and through lane

group with permitted operation (veh/h/ln), and other variables are as previously defined.

An equation for calculating g_{diff} is provided in Section 3 (Equation 31-103).

Equation 31-61 is used to compute the saturation flow rate in a shared right-turn and through lane group s_{sr}.

Equation 31-61

$$s_{sr} = \frac{s_{th}}{1 + P_R\left(E_{R,m} - 1\right)}$$

where P_R is the proportion of right-turning vehicles in the shared lane (decimal).

The saturation flow rate for the lane group serving left-turning vehicles in an exclusive lane s_l is computed with Equation 31-59, with P_L = 1.0, g_{diff} = 0.0, g_f =0.0, and s_{th} replaced by s_{lt} (see Equation 31-106). Similarly, the saturation flow rate in an exclusive right-turn lane group s_r is computed with Equation 31-61, with P_R = 1.0.

The saturation flow rate for the lane group serving through vehicles in an exclusive lane is computed with Equation 31-62.

Equation 31-62

$$s_t = s_{th} f_s$$

where f_s is the adjustment factor for all lanes serving through vehicles on an approach with a shared left-turn and through lane group (= 1.0 if N_{sl} = 0; 0.91 otherwise).

The saturation flow rate for the shared left- and right-turn lane is computed with Equation 31-63.

Equation 31-63

$$s_{lr} = \frac{s_{th}}{1 + P_L\left(E_{L,m} - 1\right) + P_R\left(E_{R,m} - 1\right)}$$

F. Compute Flow Ratio

The flow ratio for the subject intersection approach is computed with Equation 31-64.

Equation 31-64

$$y^* = \frac{v_l N_l + v_{sl} N_{sl} + v_t N_t + v_{sr} N_{sr} + v_r N_r + v_{lr} N_{lr}}{s_l N_l + s_{sl} N_{sl} + s_t N_t + s_{sr} N_{sr} + s_r N_r + s_{lr} N_{lr}}$$

where y^* is the flow ratio for the approach. If a shared left- and right-turn lane exists on the subject approach, then N_{sl} = 0, N_t = 0, N_{sr} = 0, and N_{lr} = 1; otherwise, N_{sl} = 1, $N_t \geq 0$, N_{sr} = 1, and N_{lr} = 0.

G. Compute Revised Lane Group Flow Rate

The flow ratio from Step F is used to compute the demand flow rate in the exclusive left-turn lane group with Equation 31-65.

Equation 31-65

$$v_l = s_l\, y^*$$

In a similar manner, the demand flow rate for the other lane groups is estimated by multiplying the flow ratio y^* by the corresponding lane group saturation flow rate.

H. Compute Turn-Movement Flow Rate in Shared-Lane Lane Groups

The left-turn demand flow rate in the shared lane group is computed with Equation 31-66.

$$v_{sl,lt} = v_{lt} - v_l \geq 0.0$$

Equation 31-66

Equation 31-66 can be used to compute the right-turn demand flow rate in the shared lane group by substituting $v_{sr,rt}$ for $v_{sl,lt}$, v_{rt} for v_{lt}, and v_r for v_l.

The demand flow rate in each shared-lane lane group is now compared with the rate estimated in Step B. If they differ by less than 0.1 veh/h, then the procedure is complete and the flow rates estimated in Steps G and H represent the best estimate of the flow rate for each lane group.

If there is disagreement between the lane group demand flow rates, then the calculations are repeated, starting with Step C. However, for this iteration, the flow rates computed in Steps G and H are used in the new calculation sequence. The calculations are complete when the flow rates used at the start of Step C differ from those obtained in Step H by less than 0.1 veh/h.

PRETIMED PHASE DURATION

The design of a pretimed timing plan can be a complex and iterative process that is generally carried out with the assistance of software. Several software products are available for this purpose. This subsection describes various strategies for pretimed signal timing design and provides a procedure for implementing one of these strategies.

Design Strategies

Several aspects of signal timing design are beyond the scope of this manual. One such aspect is the choice of the timing strategy. Three basic strategies are commonly used for pretimed signals.

One strategy is to equalize the volume-to-capacity ratios for critical lane groups. It is the simplest strategy and the only one that may be calculated without excessive iteration. Under this strategy, the green time is allocated among the various signal phases in proportion to the flow ratio of the critical lane group for each phase. It is described briefly in the next part. It is also used in the quick estimation method described in Section 5.

A second strategy is to minimize the total delay to all vehicles. This strategy is generally proposed as the optimal solution to the signal-timing problem. Variations of this strategy often combine other performance measures (e.g., stop rate, fuel consumption) in the optimization function. Many signal-timing software products offer this optimization feature. Some products use a delay estimation procedure identical to the methodology in Chapter 18, whereas others use minor departures from it.

A third strategy is to equalize the level of service for all critical lane groups. This strategy promotes a level of service on all approaches that is consistent with the overall intersection level of service. It improves on the first and second strategies because they tend to produce a higher delay per vehicle for the minor movements at the intersection (and therefore a less favorable level of service).

Determining Phase Duration on the Basis of Vehicle Demand

Signal timing based on equalization of the volume-to-capacity ratio is described in this part. It uses Equation 31-67, Equation 31-68, and Equation 31-69. These equations are used to estimate the cycle length and effective green time for each critical phase. Conversion to green interval duration follows by applying the appropriate lost-time increments.

Equation 31-67

$$X_c = \left(\frac{C}{C-L}\right)\sum_{i \in ci} y_{c,i}$$

Equation 31-68

$$C = \frac{L\,X_c}{X_c - \sum_{i \in ci} y_{c,i}}$$

Equation 31-69

$$g_i = \frac{v_i C}{N_i\, s_i\, X_i} = \left(\frac{v}{N\,s}\right)_i \left(\frac{C}{X_i}\right)$$

where

- C = cycle length (s),
- L = cycle lost time (s),
- X_c = critical intersection volume-to-capacity ratio,
- $y_{c,i}$ = critical flow ratio for phase $i = v_i/(N\,s_i)$,
- ci = set of critical phases on the critical path,
- X_i = volume-to-capacity ratio for lane group i,
- v_i = demand flow rate for lane group i (veh/h),
- N_i = number of lanes in lane group i (ln),
- s_i = saturation flow rate for lane group i (veh/h/ln), and
- g_i = effective green time for lane group i (s).

The summation term in each of these equations represents the summation of a specific variable for the set of critical phases. A critical phase is one phase of a set of phases that occur in sequence whose combined flow ratio is the largest for the signal cycle.

Procedure

The following steps summarize the procedure for estimating the cycle length and effective green time for the critical phases:

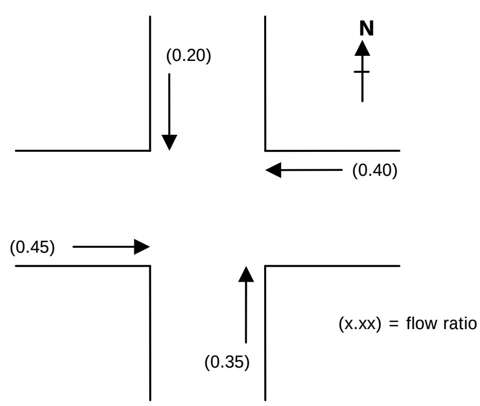
1. Compute the flow ratio [= $v_i/(Ns_i)$] for each lane group and identify the critical flow ratio for each phase. When there are several lane groups on the approach and they are served during a common phase, the lane group with the largest flow ratio represents the critical flow ratio for the phase. A procedure for identifying the critical phases and associated flow ratios is described in Chapter 18, Signalized Intersections.
2. If signal-system constraints do not dictate the cycle length, then estimate the minimum cycle length with Equation 31-68 by setting X_c equal to 1.0.

3. If signal-system constraints do not dictate the cycle length, then estimate the desired cycle length with Equation 31-68 by substituting a target volume-to-capacity ratio X_t for the critical ratio X_c. A value of X_t in the range of 0.80 to 0.90 is recommended for this purpose.
4. If signal-system constraints do not dictate the cycle length, then use the results of Steps 2 and 3 to select an appropriate cycle length for the signal. Otherwise, the cycle length is that dictated by the signal system.
5. Estimate the effective green time for each phase with Equation 31-69 and the target volume-to-capacity ratio.
6. Check the timing to ensure that the effective green time and the lost time for each phase in a common ring sum to the cycle length.

Example Application

The procedure is illustrated by a sample calculation. Consider the intersection shown in Exhibit 31-13.

Exhibit 31-13
Example Intersection



Phases 2 and 6 serve the eastbound and westbound approaches, respectively. Phases 4 and 8 serve the southbound and northbound approaches, respectively. One phase from each pair will represent the critical phase and dictate the duration of both phases. It is assumed that the lost time for each phase equals the change period (i.e., the yellow change interval plus the red clearance interval). Thus, the lost time for each critical phase is 4 s, or 8 s for the cycle.

In this simple example, only one lane group is served on each approach, so the critical flow ratios can be identified by inspection of Exhibit 31-13. Specifically, the critical flow ratio for the east–west phases is that associated with the eastbound approach (i.e., Phase 2) at a value of 0.45. Similarly, the critical flow ratio for the north–south phases is that associated with the northbound approach (i.e., Phase 8).

The minimum cycle length that will avoid oversaturation is computed by Equation 31-68 with $X_c = 1.00$.

$$C(minimum) = \frac{8(1.0)}{1.0 - (0.45 + 0.35)} = \frac{8}{0.2} = 40 \text{ s}$$

A target volume-to-capacity ratio of 0.80 is used to estimate the target cycle length.

$$C = \frac{8(0.8)}{0.8-(0.45+0.35)} = \frac{6.4}{0} = \text{infinity}$$

This computation indicates that a critical volume-to-capacity ratio of 0.8 cannot be provided with the present demand levels at the intersection.

As a second trial estimate, a target volume-to-capacity ratio of 0.92 is selected and used to estimate the target cycle length.

$$C = \frac{8(0.92)}{0.92-(0.45+0.35)} = 61 \text{ s}$$

The estimate is rounded to 60 s for practical application. Equation 31-67 is then used to estimate the critical volume-to-capacity ratio of 0.923 for the selected cycle length of 60 s.

With Equation 31-69, the effective green time is allocated so that the volume-to-capacity ratio for each critical lane group is equal to the target volume-to-capacity ratio. Thus, for the example problem, the target volume-to-capacity ratio for each phase is 0.923. The effective green times are computed with Equation 31-69. The results of the calculations are listed below:

$$g_2 = 0.45(60/0.923) = 29.3 \text{ s}$$
$$g_8 = 0.35(60/0.923) = 22.7 \text{ s}$$
$$g_2 + g_8 + L = 29.3 + 22.7 + 8.0 = 60.0 \text{ s}$$

The duration of the effective green interval for Phase 6 is the same as for Phase 2, given that they have the same phase lost time. Similarly, the effective green interval for Phase 4 is the same as for Phase 8.

Determining Phase Duration on the Basis of Pedestrian Considerations

Two pedestrian considerations are addressed in this part, as they relate to pretimed phase duration. One consideration addresses the time a pedestrian needs to perceive the signal indication and traverse the crosswalk. A second consideration addresses the time needed to serve cyclic pedestrian demand. When available, local guidelines or practice should be used to establish phase duration on the basis of pedestrian considerations.

A minimum green interval duration that allows a pedestrian to perceive the indication and traverse the crosswalk can be computed with Equation 31-70.

Equation 31-70

$$G_{p,min} = t_{pr} + \frac{L_{cc}}{S_p} - Y - R_c$$

where

$G_{p,min}$ = minimum green interval duration based on pedestrian crossing time (s),

t_{pr} = pedestrian perception of signal indication and curb departure time = 7.0 (s),

L_{cc} = curb-to-curb crossing distance (ft),

S_p = pedestrian walking speed = 3.5 (ft/s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

The variable t_{pr} in this equation represents the time pedestrians need to perceive the start of the phase and depart from the curb. A value of 7.0 s represents a conservatively long value that is adequate for most pedestrian crossing conditions. The variable S_p represents the pedestrian walking speed in a crosswalk. A value of 3.5 ft/s represents a conservatively slow value that most pedestrians will exceed.

If a permitted or protected-permitted left-turn operation is used for the left-turn movement that crosses the subject crosswalk, then the subtraction of the yellow change interval and the red clearance interval in Equation 31-70 may cause some conflict between pedestrians and left-turning vehicles. If this conflict can occur, then the minimum green interval duration should be computed as $G_{p,min} = t_{pr} + (L_{cc}/S_p)$.

The second pedestrian consideration in timing design is the time required to serve pedestrian demand. The green interval duration should equal or exceed this time to ensure pedestrian demand is served each cycle. The time needed to serve this demand is computed with either Equation 31-71 or Equation 31-72, along with Equation 31-73.

If the crosswalk width W is greater than 10 ft, then:

$$t_{ps} = 3.2 + \frac{L_{cc}}{S_p} + 2.7\frac{N_{ped}}{W}$$

Equation 31-71

If the crosswalk width W is less than or equal to 10 ft, then:

$$t_{ps} = 3.2 + \frac{L_{cc}}{S_p} + 0.27\,N_{ped}$$

Equation 31-72

with

$$N_{ped} = \frac{v_{ped,i}}{3{,}600}C$$

Equation 31-73

where

t_{ps} = pedestrian service time (s),

W = effective width of crosswalk (ft),

N_{ped} = number of pedestrians crossing during an interval (p), and

$v_{ped,i}$ = pedestrian flow rate in the subject crossing for travel direction i (p/h).

Other variables are as previously defined.

Equation 31-73 assumes that pedestrians always cross at the start of the phase. Thus, it yields a conservatively large estimate of N_{ped} because some pedestrians arrive and cross during the green indication.

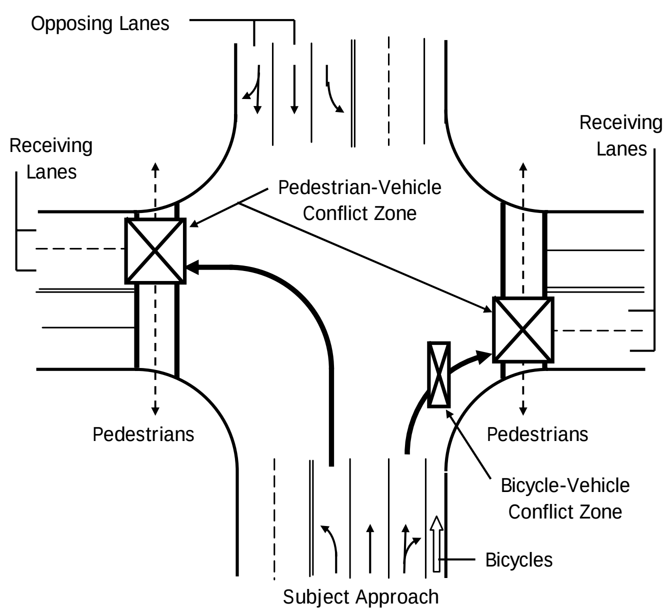
Equation 31-73 is specific to the pedestrian flow rate in one direction of travel along the subject crosswalk. If pedestrian flow rate varies significantly during the analysis period for the crosswalk's two travel directions, then t_{ps} should be calculated for both travel directions and the larger value should be used to estimate the green interval duration needed to serve pedestrian demand.

PEDESTRIAN AND BICYCLE ADJUSTMENT FACTORS

Exhibit 31-14 shows sample conflict zones where intersection users compete for space. This competition reduces the saturation flow rate of the turning automobiles. Its effect is quantified in the pedestrian and bicycle adjustment factors. This subsection describes a procedure for calculating the pedestrian and bicycle adjustment factors. These factors are used in the procedure for calculating the adjusted saturation flow rate that is described in Chapter 18.

This subsection consists of two parts. The first part describes the procedure for computing (*a*) the pedestrian–bicycle adjustment factor for right-turn lane groups and (*b*) the pedestrian adjustment factor for left-turn lane groups from a one-way street. The second part describes the procedure for computing the pedestrian adjustment factor for left-turn groups served by permitted or protected-permitted operation.

Exhibit 31-14
Conflict Zone Locations



The following guidance is used to determine the pedestrian adjustment factor for lane groups serving left-turn movements f_{Lpb}:

- If there are no conflicting pedestrians, then f_{Lpb} is equal to 1.0.
- If the lane group is on a two-way street and the protected mode or split phasing is used, then f_{Lpb} is equal to 1.0.
- If the lane group is on a two-way street and either the permitted mode or the protected-permitted mode is used, then the procedure described in the second part to follow is used to calculate f_{Lpb}.

- If the lane group is on a one-way street, then the procedure described in the first part to follow is used to compute f_{Lpb}.

The following guidance is used to determine the pedestrian–bicycle adjustment factor for lane groups serving right-turn movements f_{Rpb}:

- If there are no conflicting pedestrians or bicycles, then f_{Rpb} is equal to 1.0.
- If the protected mode is used, then f_{Rpb} is equal to 1.0.
- If the permitted mode or the protected-permitted mode is used, then the procedure described in the first part to follow is used to compute f_{Rpb}.

Right-Turn Movements and Left-Turn Movements from One-Way Street

A. Determine Pedestrian Flow Rate During Service

This procedure requires knowledge of the phase duration and cycle length. If these variables are not known and the intersection is pretimed, then they can be estimated by the quick estimation method described in Section 5. If the intersection is actuated, then the average phase duration and cycle length are computed by the procedure described previously in this section.

The pedestrian flow rate during the pedestrian service time is computed with Equation 31-74.

$$v_{pedg} = v_{ped} \frac{C}{g_{ped}} \leq 5{,}000$$

Equation 31-74

where

v_{pedg} = pedestrian flow rate during the pedestrian service time (p/h),

v_{ped} = pedestrian flow rate in the subject crossing (walking in both directions) (p/h),

C = cycle length (s), and

g_{ped} = pedestrian service time (s).

If the phase providing service to pedestrians is actuated, has a pedestrian signal head, and rest-in-walk is not enabled, then the pedestrian service time is equal to the smaller of (*a*) the effective green time for the phase or (*b*) the sum of the walk and pedestrian clear settings [i.e., $g_{ped} = \min(g, Walk + PC)$]. Otherwise, the pedestrian service time can be assumed to equal the effective green time for the phase (i.e., $g_{ped} = g$).

B. Determine Average Pedestrian Occupancy

If the pedestrian flow rate during the pedestrian service time is 1,000 p/h or less, then the pedestrian occupancy is computed with Equation 31-75.

$$OCC_{pedg} = \frac{v_{pedg}}{2{,}000}$$

Equation 31-75

where OCC_{pedg} is the pedestrian occupancy.

Alternatively, if this flow rate exceeds 1,000 p/h, then Equation 31-76 is used.

Equation 31-76

$$OCC_{pedg} = 0.4 + \frac{v_{pedg}}{10{,}000} \leq 0.90$$

A practical upper limit on v_{pedg} of 5,000 p/h should be maintained when Equation 31-76 is used.

C. Determine Bicycle Flow Rate During Green

The bicycle flow rate during the green indication is computed with Equation 31-77.

Equation 31-77

$$v_{bicg} = v_{bic}\frac{C}{g} \leq 1{,}900$$

where

v_{bicg} = bicycle flow rate during the green indication (bicycles/h),

v_{bic} = bicycle flow rate (bicycles/h),

C = cycle length (s), and

g = effective green time (s).

D. Determine Average Bicycle Occupancy

The average bicycle occupancy is computed with Equation 31-78.

Equation 31-78

$$OCC_{bicg} = 0.02 + \frac{v_{bicg}}{2{,}700}$$

where OCC_{bicg} is the bicycle occupancy and v_{bicg} is the bicycle flow rate during the green indication (bicycles/h).

A practical upper limit on v_{bicg} of 1,900 bicycles/h should be maintained when Equation 31-78 is used.

E. Determine Relevant Conflict Zone Occupancy

Equation 31-79 is used for right-turn movements with no bicycle interference or for left-turn movements from a one-way street. This equation is based on the assumptions that (*a*) pedestrian crossing activity takes place during the time period associated with g_{ped} and (*b*) no crossing occurs during the green time period $g - g_{ped}$, when this time period exists.

Equation 31-79

$$OCC_r = \frac{g_{ped}}{g} OCC_{pedg}$$

where OCC_r is the relevant conflict-zone occupancy and other variables are as previously defined.

Alternatively, Equation 31-80 is used for right-turn movements with pedestrian and bicycle interference, with all variables previously defined:

Equation 31-80

$$OCC_r = \left(\frac{g_{ped}}{g} OCC_{pedg}\right) + OCC_{bicg} - \left(\frac{g_{ped}}{g} OCC_{pedg}\ OCC_{bicg}\right)$$

F. Determine Unoccupied Time

If the number of cross-street receiving lanes is equal to the number of turn lanes, then turning vehicles will not be able to maneuver around pedestrians or bicycles. In this situation, the time the conflict zone is unoccupied is computed with Equation 31-81.

$$A_{pbT} = 1 - OCC_r$$

Equation 31-81

where A_{pbT} is the unoccupied time and OCC_r is the relevant conflict-zone occupancy.

Alternatively, if the number of cross-street receiving lanes exceeds the number of turn lanes, turning vehicles will more likely maneuver around pedestrians or bicycles. In this situation, the effect of pedestrians and bicycles on saturation flow is lower and the time the conflict zone is unoccupied is computed with Equation 31-82, with all variables as previously defined.

$$A_{pbT} = 1 - 0.6\ OCC_r$$

Equation 31-82

Either Equation 31-81 or Equation 31-82 is used to compute A_{pbT}. The choice of which equation to use should be based on careful consideration of the number of turn lanes and the number of receiving lanes. At some intersections, drivers may consistently and deliberately make illegal turns from an exclusive through lane. At other intersections, proper turning cannot be executed because the receiving lane is blocked by double-parked vehicles. For these reasons, the number of turn lanes and receiving lanes should be determined from field observation.

G. Determine Saturation Flow Rate Adjustment Factor

For permitted right-turn operation in an exclusive lane, Equation 31-83 is used to compute the pedestrian–bicycle adjustment factor.

$$f_{Rpb} = A_{pbT}$$

Equation 31-83

where f_{Rpb} is the pedestrian–bicycle adjustment factor for right-turn groups and A_{pbT} is the unoccupied time.

For protected-permitted operation in an exclusive lane, the factor from Equation 31-83 is used to compute the adjusted saturation flow rate during the permitted period. The factor has a value of 1.0 when used to compute the adjusted saturation flow rate for the protected period.

For left-turn movements from a one-way street, Equation 31-84 is used to compute the pedestrian adjustment factor.

$$f_{Lpb} = A_{pbT}$$

Equation 31-84

where f_{Lpb} is the pedestrian adjustment factor for left-turn groups and A_{pbT} is the unoccupied time.

Permitted and Protected-Permitted Left-Turn Movements

This part describes a procedure for computing the adjustment factor for left-turn movements on a two-way street that are operating in either the permitted

mode or the protected-permitted mode. The calculations in this part supplement the procedure described in the previous part. The calculations described in Steps A and B in the previous part must be completed first (substitute the effective permitted green time g_p for g in Step A). Then, the calculations described in this part are completed.

This procedure does not account for vehicle–bicycle conflict during the left-turn maneuver.

A. Compute Pedestrian Occupancy After Queue Clears

The pedestrian occupancy after the opposing queue clears is computed with Equation 31-85 or Equation 31-86. The opposing-queue service time g_q is computed as the effective permitted green time g_p less the duration of permitted left-turn green time that is not blocked by an opposing queue g_u (i.e., $g_q = g_p - g_u$).

If $g_q < g_{ped}$ then:

Equation 31-85

$$OCC_{pedu} = OCC_{pedg}\left(1 - 0.5\ g_q / g_{ped}\right)$$

otherwise:

Equation 31-86

$$OCC_{pedu} = 0.0$$

where OCC_{pedu} is the pedestrian occupancy after the opposing queue clears, g_q is the opposing queue service time (= g_s for the opposing movement) (s), and other variables are as previously defined.

If the opposing-queue service time g_q equals or exceeds the pedestrian service time g_{ped}, then the opposing queue consumes the entire pedestrian service time.

B. Determine Relevant Conflict Zone Occupancy

After the opposing queue clears, left-turning vehicles complete their maneuvers on the basis of accepted gap availability in the opposing traffic stream. Relevant conflict-zone occupancy is a function of the probability of accepted gap availability and pedestrian occupancy. It is computed with Equation 31-87.

Equation 31-87

$$OCC_r = \frac{g_{ped} - g_q}{g_p - g_q}\left(OCC_{pedu}\right)e^{-5.00\ v_0 / 3{,}600}$$

where v_o is the opposing demand flow rate (veh/h), g_p is the effective green time for permitted left-turn operation (s), and other variables are as previously defined.

C. Determine Unoccupied Time

Either Equation 31-81 or Equation 31-82 from the previous part (i.e., Step F) is used to compute A_{pbT}. The choice of which equation to use should be based on consideration of the number of left-turn lanes and the number of receiving lanes.

D. Determine Saturation Flow Rate Adjustment Factor

Equation 31-88 is used to compute the pedestrian adjustment factor f_{Lpb} from A_{pbT}, the unoccupied time.

$$f_{Lpb} = A_{pbT}$$

Equation 31-88

3. QUEUE ACCUMULATION POLYGON

INTRODUCTION

This section describes a procedure for using the queue accumulation polygon (QAP) concept. It consists of three main subsections. The first subsection provides a review of concepts related to the QAP. The second subsection describes a general procedure for developing the QAP. The third subsection extends the general procedure to the evaluation of left-turn lane groups.

The discussion in this section describes basic principles for developing polygons for selected types of lane assignment, lane grouping, left-turn operation, and phase sequence. The analyst is referred to the computational engine (see Section 7) for specific calculation details, especially as they relate to assignments, groupings, left-turn operations, and phase sequences not addressed in this section.

CONCEPTS

The QAP is a graphic tool for describing the deterministic relationship between vehicle arrivals, departures, queue service time, and delay. The QAP defines the queue size for a traffic movement as a function of time during the cycle. The shape of the polygon is defined by the following factors: arrival flow rate during the effective red and green intervals, saturation flow rate associated with each movement in the lane group, signal indication status, left-turn operation mode, and phase sequence. Once constructed, the polygon can be used to compute the queue service time and uniform delay for the corresponding lane group.

A QAP is shown in Exhibit 31-15. The variables shown in the exhibit are defined in the following list:

r = effective red time = $C - g$ (s),

g = effective green time (s),

C = cycle length (s),

g_s = queue service time = $Q_r/(s - q_g)$ (s),

g_e = green extension time (s),

q = arrival flow rate = $v/3{,}600$ (veh/s),

v = demand flow rate (veh/h),

q_r = arrival flow rate during the effective red time = $(1 - P)qC/r$ (veh/s),

q_g = arrival flow rate during the effective green time = PqC/g (veh/s),

Q_r = queue size at the end of the effective red time = $q_r r$ (veh),

P = proportion of vehicles arriving during the green indication (decimal), and

s = adjusted saturation flow rate (veh/h/ln).

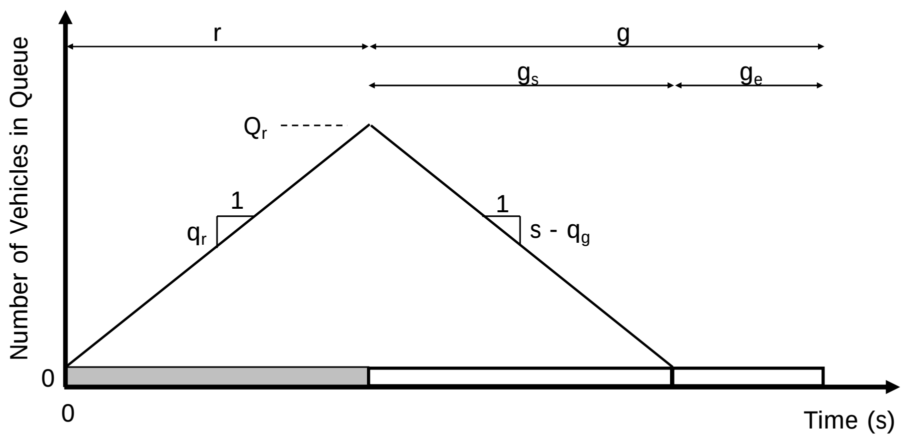


Exhibit 31-15
Queue Accumulation Polygon for Protected Movements

In application, all flow rate variables are converted to common units of vehicles per second per lane. The presentation in this section is based on these units for *q* and *s*.

The polygon in Exhibit 31-15 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. Other polygon shapes are possible, depending on whether the lane group includes a shared lane and whether the lane group serves a permitted (or protected-permitted) left-turn movement. In general, a unique polygon shape will be dictated by each combination of left-turn operational mode (i.e., permitted, protected, protected-permitted) and phase sequence (i.e., lead, lag, split). A general procedure for constructing these polygons is described in the next subsection.

GENERAL PROCEDURE

This subsection describes a general procedure for constructing the QAP for a lane group at a signalized intersection. It is directly applicable to left-turn lane groups that have exclusive lanes and protected phasing, through lane groups with exclusive lanes, and right-turn lane groups with exclusive lanes. Variations of this procedure that extend it to turn lane groups with shared lanes, permitted operation, or protected-permitted operation are described in the next subsection.

The construction of a QAP is based on identification of flow rates and service times during the average signal cycle. These rates and times define periods of queue growth, queue service, and service upon arrival. As shown in Exhibit 31-15, the rates and times define queue size as it varies during the cycle. The resulting polygon formed by the queue size profile can be decomposed into a series of trapezoid or triangle shapes, with each shape having a known time interval. Collectively, the areas of the individual shapes can be added to equal the area of the polygon and the time intervals added to equal the cycle length.

The QAP calculation sequence follows the order of interval occurrence over time, and the results can be recorded graphically (e.g., Exhibit 31-15) or in a tabular manner (i.e., row by row, where each row represents one time interval). A time interval is defined to begin and end at points where either the departure rate or the arrival rate changes. For the duration of the interval, these rates are assumed to be constant.

The following text outlines the calculation sequence used to construct a QAP for a specified lane group. The sequence is repeated for each lane group at the intersection, with the through lane groups evaluated first so that the saturation flow rate of permitted left-turn lane groups can be based on the known queue service time for the opposing traffic movements.

1. The QAP calculations for a given lane group start with the end of the effective green period for the phase serving the subject lane group in a protected manner. The initial queue Q_i is assumed to equal 0.0 veh.
2. Determine the points in the cycle where the arrival flow rate or the discharge rate changes. The arrival rate may change because of platoons formed by an upstream signal, so it is expressed in terms of the arrival rate during green q_g and during red q_r. The discharge rate may change because of the start or end of effective green, a change in the saturation flow rate, the depletion of the subject queue, the depletion of the opposing queue, or the departure of left-turn vehicles as sneakers.
3. For the time interval between the points identified in Step 2, number each interval and compute its duration. Also, identify the arrival rate and discharge rate associated with the interval. Finally, confirm that the sum of all interval durations equals the cycle length.
4. Calculate the capacity of each interval for which there is some discharge, including sneakers when applicable. The sum of these capacities equals the total lane group capacity. Calculate the demand volume for each interval for which there are some arrivals. The sum of these volumes equals the total lane group volume.
5. Calculate the volume-to-capacity ratio X for the lane group by dividing the lane group's total volume by its total capacity. If the volume-to-capacity ratio exceeds 1.0, then calculate the adjusted arrival flow rate q' for each interval by dividing the original flow rate q by X (i.e., $q' = q/X$).
6. Calculate the queue at the end of interval i with Equation 31-89.

Equation 31-89

$$Q_i = Q_{i-1} - (s/3{,}600 - q/N)\, t_{d,i} \geq 0.0$$

where Q_i is the queue size at the end of interval i (veh), $t_{d,I}$ is the duration of time interval i during which the arrival flow rate and saturation flow rate are constant (s), and other variables are as previously defined.

7. If the queue at the end of interval i equals 0.0 veh, then compute the duration of the trapezoid or triangle with Equation 31-90. The subject interval should be divided into two intervals, with the first interval having a duration of $t_{t,i}$ and the second interval having a duration of $t_{d,i} - t_{t,i}$. The second interval has starting and ending queues equal to 0.0 veh.

Equation 31-90

$$t_{t,i} = \min(t_{d,i}, Q_{i-1}/w_q)$$

where $t_{t,i}$ is the duration of trapezoid or triangle in interval i (s), w_q is the queue change rate (= discharge rate minus arrival rate) (veh/s), and other variables are as previously defined.

8. Steps 6 and 7 are repeated for each interval in the cycle.

9. When all intervals are completed, the assumption of a zero starting queue (made in Step 1) is checked. The queue size computed for the last interval should always equal the initially assumed value. If this is not the case, then Steps 6 through 8 are repeated by using the ending queue size of the last interval as the starting queue size for the first interval.
10. When all intervals have been evaluated and the starting and ending queue sizes are equal, then the uniform delay can be calculated. This calculation starts with computing the area of each trapezoid or triangle. Then, these areas are added to determine the total delay. Finally, the total delay is divided by the number of arrivals per cycle to produce uniform delay. Equations for calculating uniform delay by using the QAP are described in Step 7 of the next subsection.

PROCEDURE FOR SELECTED LANE GROUPS

This subsection describes a seven-step procedure for constructing the QAP for selected lane groups. The focus is on left-turn movements in lane groups with shared lanes, permitted operation, or protected-permitted operation. However, there is some discussion of other lane groups, lane assignments, and operation. The procedure described in this subsection represents an extension of the general procedure described in the previous subsection.

Step 1. Determine Permitted Green Time

This step applies when the subject left-turn movement is served by using the permitted mode or the protected-permitted mode. Two effective green times are computed. One is the effective green time for permitted left-turn operation g_p. This green time occurs during the period when the adjacent and opposing through movements both have a green ball indication (after adjustment for lost time).

The other effective green time represents the duration of permitted left-turn green time that is not blocked by an opposing queue g_u. This green time represents the time during the effective green time for permitted left-turn operation g_p that is not used to serve the opposing queue. This time is available to the subject left-turn movement to filter through the conflicting traffic stream.

Exhibit 31-16 provides equations for computing the unblocked permitted green time for left-turn Movement 1 (see Exhibit 31-1) when Dallas left-turn phasing is not used. Similar equations can be derived for the other left-turn movements or when Dallas phasing is used. The variables defined in this exhibit are provided in the following list:

g_u = duration of permitted left-turn green time that is not blocked by an opposing queue (s),

G_U = displayed green interval corresponding to g_u (s),

e = extension of effective green = 2.0 (s),

l_1 = start-up lost time = 2.0 (s),

G_q = displayed green interval corresponding to g_q (s),

D_p = phase duration (s),

R_c = red clearance interval (s),

Y = yellow change interval (s), and

g_q = opposing queue service time (= g_s for the opposing movement) (s).

Exhibit 31-16
Unblocked Permitted Green Time

Phase Sequence (phase numbers shown in boxes)		Displayed Unblocked Permitted Green Time G_U (s)[a]	Permitted Start-Up Lost Time $l_{1,p}$ (s)[b]	Permitted Extension Time e_p (s)[c]
Lead-Lead	1 2 / 5 6	$G_{U1} = \min[D_{p1} + D_{p2} - D_{p5} - Y_6 - R_{c6}, G_{U1}^*]$ with $G_{U1}^* = D_{p2} - Y_6 - R_{c6} - G_{q2}$	$l_{1,1}^*$	e_1
	1 2 / 5 6	$G_{U1} = D_{p2} - Y_6 - R_{c6} - G_{q2}$	$l_{1,1}^*$	e_1
Lead-Lag or Lead-Perm	1 2 / 6 5	$G_{U1} = D_{p6} - Y_6 - R_{c6} - D_{p1} - G_{q2}$	0.0	e_1
	1 2 / 6 5	No permitted period	Not applicable	Not applicable
	1 2 / 6	$G_{U1} = D_{p6} - Y_6 - R_{c6} - D_{p1} - G_{q2}$	0.0	e_1
Lag-Lead or Lag-Perm	2 1 / 5 6	No permitted period	Not applicable	Not applicable
	2 1 / 5 6	$G_{U1} = D_{p2} - Y_2 - R_{c2} - \max[D_{p5}, G_{q2}]$	$l_{1,1}$	0.0
	2 1 / 6	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2}, D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	0.0
Perm-Lead	2 / 5 6	$G_{U1} = D_{p2} - Y_2 - R_{c2} - \max[D_{p5}, G_{q2}]$	$l_{1,1}$	e_1
Perm-Lag	2 / 6 5	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2}, D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	e_1
Perm-Perm	2 / 6	$G_{U1} = D_{p2} - Y_6 - R_{c6} - G_{q2}$	$l_{1,1}$	e_1
Lag-Lag	2 1 / 6 5	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2}, D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	e_1^*
	2 1 / 6 5	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2}, D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	e_1^*

Notes: [a] G_{q2} is computed for each opposing lane and the value used corresponds to the lane requiring the longest time to clear. In general, if the opposing lanes serve through movements exclusively, then $G_{q2} = g_q + l_1$. If an opposing lane is shared, then $G_{q2} = g_p - g_e + l_1$, where g_p is the effective green time for permitted operation (s), g_e is the green extension time (s), and l_1 is the start-up lost time (s).
[b] If $D_{p5} > (D_{p1} - Y_1 - R_{c1})$ then, $l_1^* = D_{p5} - (D_{p1} - Y_1 - R_{c1}) + l_1 - e_1$; otherwise, $l_1^* = 0.0$. Regardless, the result should not be less than 0.0 or more than l_1.
[c] $e_1^* = D_{p2} - (D_{p6} - Y_6 - R_{c6})$, provided that the result is not less than 0.0 or more than e_1.
Perm = permitted.

For the first four variables in the preceding list, the subscript "1" is added to the variable when used in Exhibit 31-16. This subscript denotes Movement 1. For the next four variables in the list, a numeric subscript is added to the variable when used in the exhibit. This subscript denotes the phase number associated with the variable. Exhibit 31-16 applies only to left-turn Movement 1. The subscripts need to be changed to apply the equations to other left-turn movements.

The equations shown in Exhibit 31-16 indicate that the effective green time for the permitted operation of Phase 1 depends on the duration of Phase 2 and

sometimes the duration of Phase 5. In all instances, Movement 1 has permitted operation during all, or a portion of, Phase 6.

For a given left-turn lane group, one of the equations in the second column of Exhibit 31-16 will apply. It is used to compute the displayed green interval corresponding to g_u (i.e., G_U). The computed G_u is required to have a nonnegative value. If the calculation yields a negative value, then G_u is set to 0.0.

The same equation can be used to compute the displayed green interval corresponding to g_p (i.e., G_p) by substituting G_p for G_U and 0.0 for G_q. Again, the computed G_p is required to have a nonnegative value. If the calculation yields a negative value, then G_p is set to 0.0.

Equation 31-91 is used to compute the effective green time for permitted left-turn operation.

$$g_p = G_p - l_{1,p} + e_p \geq 0.0$$

Equation 31-91

where

- g_p = effective green time for permitted left-turn operation (s),
- G_p = displayed green interval corresponding to g_p (s),
- $l_{1,p}$ = permitted start-up lost time (s), and
- e_p = permitted extension of effective green (s).

The values of $l_{1,p}$ and e_p used in Equation 31-91 are obtained from columns three and four, respectively, of Exhibit 31-16.

The start-up lost time for g_u is considered to occur coincident with the start-up lost time associated with g_p. Hence, if the opposing queue service time consumes an initial portion of g_p, then there is no start-up lost time associated with g_u. The rationale for this approach is that left-turn drivers waiting for the opposing queue to clear will be anticipating queue clearance and may be moving forward slowly (perhaps already beyond the stop line) so that there is negligible start-up lost time at this point. This approach also accommodates the consideration of multiple effective green time terms when there is a shared lane (e.g., g_f) and it avoids inclusion of multiple start-up lost times during g_p. In accordance with this rationale, Equation 31-92 is used to compute the permitted left-turn green time that is not blocked by an opposing queue g_u, where other variables are as previously defined.

$$g_u = G_u + e_p \leq g_p$$

Equation 31-92

If protected-permitted operation exists and Dallas phasing is used, then the displayed green interval corresponding to g_u (i.e., G_U) is equal to the opposing through phase duration minus the queue service time and change period of the opposing through phase (i.e., $G_{U1} = D_{p2} - Y_2 - R_{c2} - G_{q2}$). The permitted start-up lost time $l_{1,p}$ and permitted extension of effective green e_p are equal to l_1 and e, respectively. Otherwise, all the calculations described previously apply.

Step 2. Determine Time Before First Left-Turn Vehicle Arrives

This step applies when the left-turn movement is served by using the permitted mode on a shared-lane approach. The variable of interest represents

the time that elapses from the start of the permitted green to the arrival of the first left-turning vehicle at the stop line. During this time, through vehicles in the shared lane are served at the saturation flow rate of an exclusive through lane.

Considerations of vehicle distribution impose an upper limit on the time before the first left-turn vehicle arrives when it is used to define a period of saturation flow. This limit is computed with Equation 31-93.

Equation 31-93

$$g_{f,\max} = \frac{(1 - P_L)}{0.5\ P_L}\left(1 - [1 - P_L]^{0.5 g_p}\right) - l_{1,p} \geq 0.0$$

where $g_{f,max}$ is the maximum time before the first left-turning vehicle arrives and within which there are sufficient through vehicles to depart at saturation (s), P_L is the proportion of left-turning vehicles in the shared lane (decimal), and other variables are as previously defined.

The value of 0.5 in two locations in Equation 31-93 represents the approximate saturation flow rate (in veh/s) of through vehicles in an exclusive lane. This approximation simplifies the calculation and provides sufficient accuracy in the estimate of $g_{f,max}$.

The time before the first left-turning vehicle arrives and blocks the shared lane is computed with Equation 31-94 or Equation 31-95, along with Equation 31-96.

If the approach has one lane, then:

Equation 31-94

$$g_f = \max(G_p\ e^{-0.860\ LTC^{0.629}} - l_{1,p}, 0.0) \leq g_{f,\max}$$

otherwise:

Equation 31-95

$$g_f = \max(G_p\ e^{-0.882\ LTC^{0.717}} - l_{1,p}, 0.0) \leq g_{f,\max}$$

with

Equation 31-96

$$LTC = \frac{v_{lt}\ C}{3{,}600}$$

where

g_f = time before the first left-turning vehicle arrives and blocks the shared lane (s),

LTC = left-turn flow rate per cycle (veh/cycle), and

v_{lt} = left-turn demand flow rate (veh/h).

Other variables are as previously defined.

Step 3. Determine Permitted Left-Turn Saturation Flow Rate

This step applies when left-turning vehicles are served by using the permitted mode or the protected-permitted mode from an exclusive lane. The saturation flow rate for permitted left-turn operation is calculated with Equation 31-97.

$$s_p = \frac{v_o\, e^{-v_o\, t_{cg}/3{,}600}}{1 - e^{-v_o\, t_{fh}/3{,}600}}$$

Equation 31-97

where

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln);

v_o = opposing demand flow rate (veh/h);

t_{cg} = critical headway = 4.5 (s); and

t_{fh} = follow-up headway (4.5 if the subject left turn is served in a shared lane, 2.5 if the subject left turn is served in an exclusive lane) (s).

In those instances in which the opposing volume equals 0.0 veh/h during the analysis period, the opposing volume is set to a value of 0.1 veh/h.

The opposing demand flow rate is not adjusted for unequal lane use in this equation. Increasing this flow rate to account for unequal lane use would misrepresent the frequency and size of headways in the opposing traffic stream. Thus, this adjustment would result in the left-turn saturation flow rate being underestimated.

Step 4. Determine Through-Car Equivalent

This step applies when left-turning vehicles are served by using the permitted mode or the protected-permitted mode. Two variables are computed to quantify the relationship between left-turn saturation flow rate and the base saturation flow rate. The first variable represents the more common case in which left-turning vehicles filter through an oncoming traffic stream. It is computed from Equation 31-98.

$$E_{L1} = \frac{s_o}{s_p}$$

Equation 31-98

where

E_{L1} = equivalent number of through cars for a permitted left-turning vehicle,

s_o = base saturation flow rate (pc/h/ln), and

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln).

The second variable to be computed represents the case in which the opposing approach has one lane. It describes the saturation flow rate during the time interval coincident with the queue service time of the opposing queue. For this case, the saturation flow rate during the period after the arrival of the first blocking left-turning vehicle and before the end of the opposing queue service time is influenced by the proportion of left-turning vehicles in the opposing traffic stream. These vehicles create artificial gaps in the opposing traffic stream through which the blocking left-turning vehicles on the subject approach can turn. This effect is considered through calculation of the following through-car equivalency factor with Equation 31-99 and Equation 31-100.

Equation 31-99

$$E_{L2} = \frac{1-(1-P_{lto})^{n_q}}{P_{lto}} \geq 1.0$$

with

Equation 31-100

$$n_q = 0.5\,(g_p - g_u - g_f) \geq 0.0$$

where

E_{L2} = equivalent number of through cars for a permitted left-turning vehicle when opposed by a queue on a single-lane approach,

P_{lto} = proportion of left-turning vehicles in the opposing traffic stream (decimal), and

n_q = maximum number of opposing vehicles that could arrive after g_f and before g_u (veh).

Other variables are as previously defined.

Step 5. Determine Proportion of Turns in a Shared Lane

This step applies when turning vehicles share a lane with through vehicles and the approach has two or more lanes. The proportion of turning vehicles in the shared lane is used in the next step to determine the saturation flow rate for the shared lane.

The proportion of left-turning vehicles in the shared lane P_L is computed if the shared lane includes left-turning vehicles. The proportion of right-turning vehicles in the shared lane P_R is computed if the shared lane includes right-turning vehicles. Guidance for computing these two variables is provided in Section 2.

If the approach has one traffic lane, then P_L equals the proportion of left-turning vehicles on the subject approach P_{lt} and P_R equals the proportion of right-turning vehicles on the subject approach P_{rt}.

Step 6. Determine Lane Group Saturation Flow Rate

The saturation flow rate for the lane group is computed during this step. When the lane group consists of an exclusive lane operating in the protected mode, then it has one saturation flow rate. This rate equals the adjusted saturation flow rate computed by the procedure described in the automobile methodology in Chapter 18.

The focus of discussion in this step is the calculation of saturation flow rate for shared-lane lane groups and for lane groups for which the permitted or protected-permitted mode is used. As the discussion indicates, these lane groups often have two or more saturation flow rates, depending on the phase sequence and operational mode of the turn movements.

Permitted Right-Turn Operation in Shared Lane

The saturation flow rate for permitted right-turn operation in a shared lane is computed with Equation 31-101.

$$s_{sr} = \frac{s_{th}}{1 + P_R\left(\frac{E_R}{f_{Rpb}} - 1\right)}$$

Equation 31-101

where

s_{sr} = saturation flow rate in shared right-turn and through lane group with permitted operation (veh/h/ln),

s_{th} = saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, and area type) (veh/h/ln),

P_R = proportion of right-turning vehicles in the shared lane (decimal),

E_R = equivalent number of through cars for a protected right-turning vehicle = 1.18, and

f_{Rpb} = pedestrian–bicycle adjustment factor for right-turn groups.

The value of f_{Rpb} is obtained by the procedure described in Section 2.

The saturation flow rate for the permitted period of a protected-permitted operation in a shared lane is equal to that obtained from Equation 31-101. The saturation flow rate for the protected period is equal to that obtained from the equation when f_{Rpb} is equal to 1.0.

Permitted Left-Turn Operation in Exclusive Lane

The saturation flow rate for a permitted left-turn operation in an exclusive lane is computed with Equation 31-102.

$$s_l = s_p f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{Lpb}$$

Equation 31-102

where s_l is the saturation flow rate in an exclusive left-turn lane group with permitted operation (veh/h/ln) and the other variables are defined following Equation 18-5 in Chapter 18.

Permitted Left-Turn Operation in Shared Lane

There are three possible saturation flow periods during the effective green time associated with permitted left-turn operation. The first period occurs before arrival of the first left-turning vehicle in the shared lane. This left-turning vehicle will block the shared lane until the opposing queue clears and a gap is available in the opposing traffic stream. The duration of this flow period is g_f. The saturation flow during this period is equal to s_{th}.

The second period of flow begins after g_f and ends with clearance of the opposing queue. It is computed with the following equation:

$$g_{diff} = g_p - g_u - g_f \geq 0.0$$

Equation 31-103

where g_{diff} is the supplemental service time for shared single-lane approaches (s) and other variables are as previously defined. This period may or may not exist, depending on the values of g_u and g_f.

If there are two or more opposing traffic lanes, then the saturation flow during the second period s_{sl2} equals 0.0 veh/h/ln. However, if the opposing approach has only one traffic lane, then the flow during this period occurs at a reduced rate that reflects the blocking effect of left-turning vehicles as they await an opposing left-turning vehicle. Left-turning vehicles during this period are assigned a through-car equivalent E_{L2}. The saturation flow rate for the shared lane is computed with Equation 31-104.

Equation 31-104

$$s_{sl2} = \frac{s_{th}}{1 + P_L\left(\dfrac{E_{L2}}{f_{Lpb}} - 1\right)}$$

where s_{sl2} is the saturation flow rate in the shared left-turn and through lane group during Period 2 (veh/h/ln) and P_L is the proportion of left-turning vehicles in the shared lane (decimal).

The third period of flow begins after clearance of the opposing queue or arrival of the first blocking left-turn vehicle, whichever occurs last. Its duration equals the smaller of $g_p - g_f$ or g_u. The saturation flow rate for this period is computed with Equation 31-105.

Equation 31-105

$$s_{sl3} = \frac{s_{th}}{1 + P_L\left(\dfrac{E_{L1}}{f_{Lpb}} - 1\right)}$$

where s_{sl3} is the saturation flow rate in the shared left-turn and through lane group during Period 3 (veh/h/ln).

For multiple-lane approaches, the impact of the shared lane is extended to include the adjacent through traffic lanes. Specifically, queued drivers are observed to maneuver from lane to lane on the approach to avoid delay associated with the left-turning vehicles in the shared lane. The effect of this impact is accounted for by multiplying the saturation flow rate of the adjacent lanes by a factor of 0.91.

Protected-Permitted Left-Turn Operation in Exclusive Lane

Two saturation flow rates are associated with protected-permitted operation. The saturation flow rate during the protected period s_{lt} is computed with Equation 31-106.

Equation 31-106

$$s_{lt} = s_o f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT}$$

where s_{lt} is the saturation flow rate of an exclusive left-turn lane with protected operation (veh/h/ln) and the other variables are defined following Equation 18-5 in Chapter 18.

The saturation flow rate during the permitted period is computed with Equation 31-102. The duration of the permitted period is equal to g_u.

Protected-Permitted Left-Turn Operation in Shared Lane

The use of a protected-permitted operation in a shared lane has some special requirements to ensure safe and efficient operation. This operational mode requires display of the green ball when the left-turn green arrow is displayed (i.e., the green arrow is not displayed without also displaying the green ball). The following conditions are applied for actuated, protected-permitted operation in a shared lane:

- The left-turn phase is set to minimum recall.
- The maximum green setting for the left-turn phase must be less than or equal to the minimum green for the adjacent through phase.
- If both opposing approaches have protected-permitted operation in a shared lane, then the phase sequence must be lead–lag.
- No vehicle detection is assigned to the left-turn phase.
- Vehicle detection in the shared lane is assigned to the adjacent through movement phase.

There are four possible saturation flow periods during the effective green time associated with protected-permitted left-turn operation in a shared lane. The first three periods are the same as those for permitted left-turn operation in a shared lane (as described previously).

The fourth period of flow coincides with the left-turn phase (i.e., the protected period). Its duration is equal to the effective green time for the left-turn phase g_l. The flow rate during this period is computed with Equation 31-107.

$$s_{sl4} = \frac{s_{th}}{1 + P_L\left(E_L - 1\right)}$$

Equation 31-107

where s_{sl4} is the saturation flow rate in the shared left-turn and through lane group during Period 4 (veh/h/ln) and other variables are as previously defined.

For multiple-lane approaches, the impact of the shared lane is extended to include the adjacent through lanes. This impact is accounted for by multiplying the saturation flow rate of the adjacent lanes by a factor of 0.91.

Protected Left- and Right-Turn Operation in a Shared Lane

The saturation flow rate in a shared left- and right-turn lane group with protected operation is computed with Equation 31-108.

$$s_{lr} = \frac{s_{th}}{1 + P_L\left(E_L - 1\right) + P_R\left(E_R - 1\right)}$$

Equation 31-108

where s_{lr} is the saturation flow rate in the shared left- and right-turn lane group (veh/h/ln).

Step 7. Define Queue Accumulation Polygon

During this step, the green times and saturation flow rates are used to construct the QAP associated with each lane group. The polygon is then used to estimate uniform delay and queue service time. With regard to the latter item,

the lane group with the longest queue service time dictates the queue service time for the phase.

The QAP in Exhibit 31-15 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. This polygon also applies to split phasing and to shared lane groups serving through and right-turning vehicles operating with the permitted mode. For split phasing, each approach is evaluated separately to determine its queue service time and uniform delay. If the approach has left- or right-turn lanes, then a separate polygon is constructed for each turn lane group.

More complicated combinations of lane assignment, phase sequence, and left-turn operational mode dictate more complicated polygons. A polygon (or its tabular equivalent) must be derived for each combination. The most common combinations are illustrated in Exhibit 31-17 to Exhibit 31-20.

The concept is extended to shared left-turn and through lane groups with protected-permitted operation in Exhibit 31-21 and Exhibit 31-22. Other polygon shapes exist, depending on traffic flow rates, phase sequence, lane use, and left-turn operational mode. The concept of polygon construction must be extended to these other combinations to accurately estimate queue service time and uniform delay.

Most of the variables shown in the following exhibits were defined in a previous subsection. The following variables are also defined at this time:

g_l = effective green time for left-turn phase (s);

g_{ps} = queue service time during permitted left-turn operation (s);

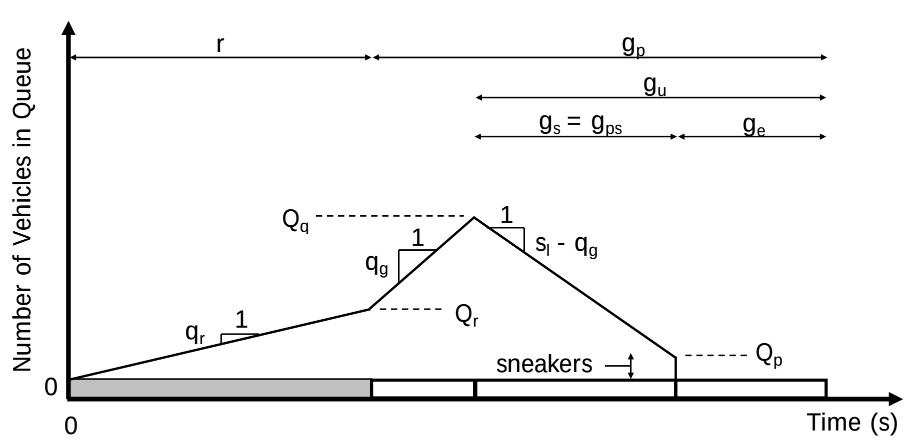
Q_q = queue size at the start of g_u (veh);

Q_p = queue size at the end of permitted service time (veh);

Q'_p = queue size at the end of permitted service time, adjusted for sneakers (veh); and

Q_f = queue size at the end of g_f (veh).

Exhibit 31-17
QAP for Permitted Left-Turn Operation in an Exclusive Lane



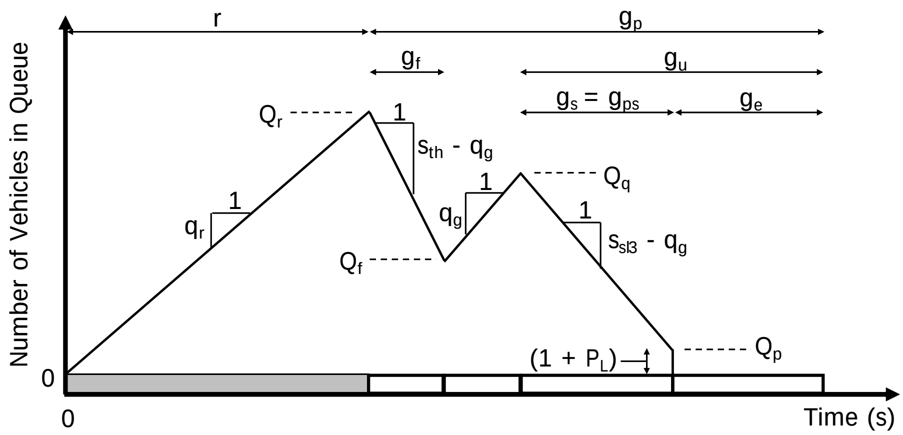


Exhibit 31-18
QAP for Permitted Left-Turn Operation in a Shared Lane

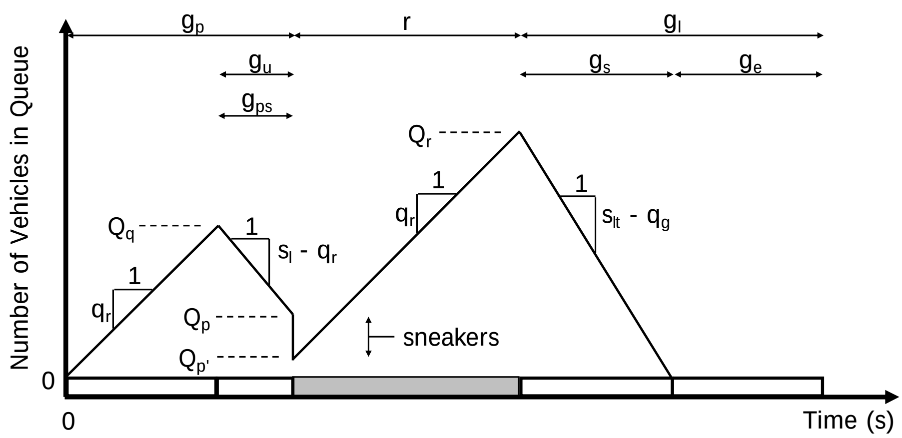


Exhibit 31-19
QAP for Leading, Protected-Permitted Left-Turn Operation in an Exclusive Lane

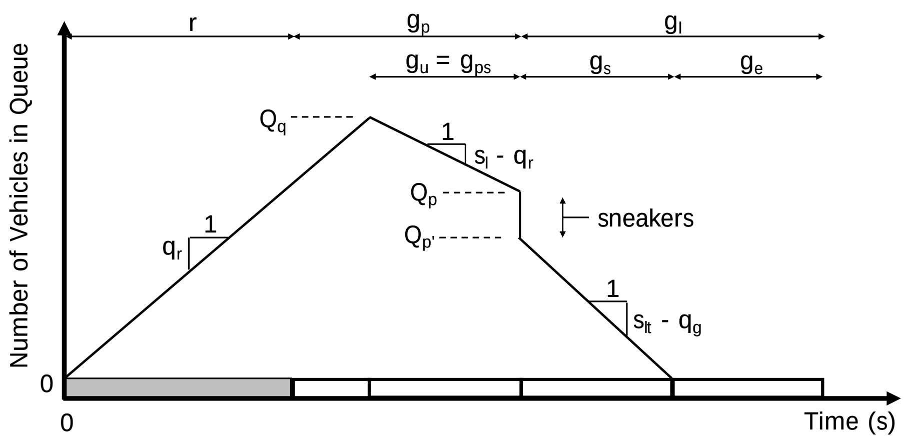


Exhibit 31-20
QAP for Lagging, Protected-Permitted Left-Turn Operation in an Exclusive Lane

Exhibit 31-21
QAP for Leading, Protected-Permitted Left-Turn Operation in a Shared Lane

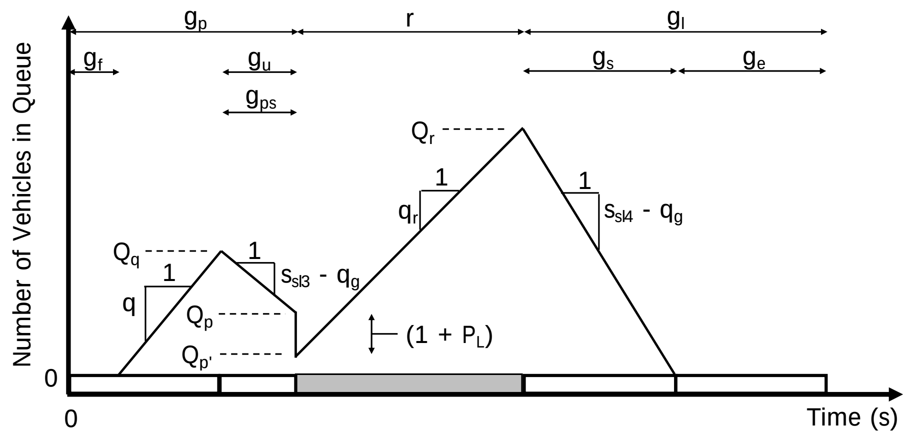
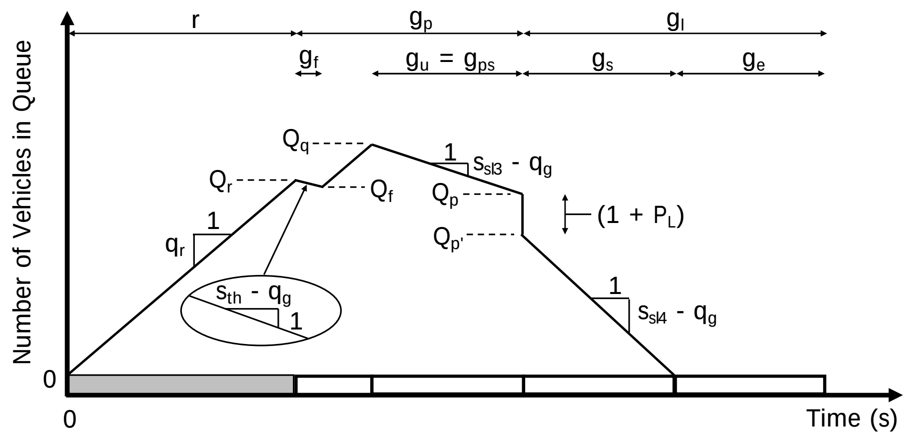


Exhibit 31-22
QAP for Lagging, Protected-Permitted Left-Turn Operation in a Shared Lane



The polygon in Exhibit 31-17 applies to the left-turn lane group with an exclusive lane that operates in the permitted mode during the adjacent through phase. If the phase extends to max out, then some left-turning vehicles will be served as sneakers.

The polygon in Exhibit 31-18 applies to the left-turn and through lane group on a shared lane approach with permitted operation. If the phase extends to max out, then some left-turning vehicles will be served as sneakers. The expected number of sneakers served is computed as $1 + P_L$, where P_L is the proportion of left-turning vehicles in the shared lane.

The polygon in Exhibit 31-19 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and an exclusive lane. The polygon in Exhibit 31-20 applies to left-turn movements that have protected-permitted operation with a lagging left-turn phase and an exclusive lane. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the number of sneakers.

The polygon in Exhibit 31-21 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and a shared left-turn and through lane group. The polygon in Exhibit 31-22 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the expected number of sneakers served (which is computed as $1 + P_L$).

As noted previously, all polygons are based on the requirement that lane volume cannot exceed lane capacity for the purpose of estimating the queue service time. This requirement is met in the polygons shown because the queue size equals 0.0 veh at some point during the cycle.

Exhibit 31-18 through Exhibit 31-22 are shown to indicate that queue size equals 0.0 veh at the start of the cycle (i.e., time = 0.0 s). In fact, the queue may not equal 0.0 veh at the start of the cycle for these polygons. Rather, there may be a nonzero queue at the start of the cycle and a queue of 0.0 veh may not be reached until a different time in the cycle. Thus, in modeling any one of the polygons in Exhibit 31-18 through Exhibit 31-22, an iterative process is required. For the first iteration, the queue is assumed to equal 0.0 veh at the start of the cycle. The polygon is then constructed and the queue status is checked at the end of the cycle. If the queue at the end of the cycle is not 0.0 veh, then this value is used as a starting point in a second polygon construction. The second polygon will result in a queue at the end of the cycle that equals the queue used at the start of the cycle. Moreover, a queue value of 0.0 veh will occur at some point in the cycle.

A. Compute Uniform Delay and Queue Service Time

The procedure for calculating uniform delay and queue service time is described in this step. Exhibit 31-23 is used for this purpose.

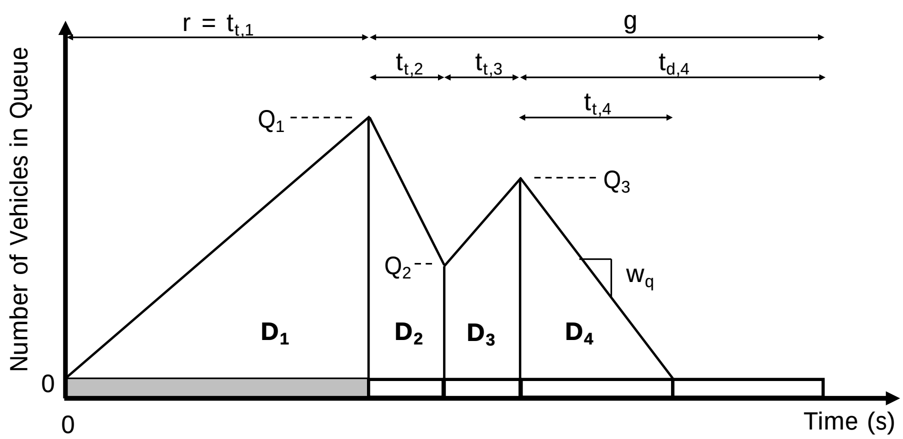


Exhibit 31-23
Polygon for Uniform Delay Calculation

The area bounded by the polygon represents the total delay incurred during the average cycle. The total delay is then divided by the number of arrivals per cycle to estimate the average uniform delay. These calculations are summarized in Equation 31-109 with Equation 31-110.

$$d_1 = \frac{0.5\sum_{i=1}(Q_{i-1} + Q_i)\, t_{t,i}}{qC}$$

Equation 31-109

with

$$t_{t,i} = \min(t_{d,i}, Q_{i-1} / w_q)$$

Equation 31-110

where d_1 is the uniform delay (s/veh) and other variables are as previously defined.

The summation term in Equation 31-109 includes all intervals for which there is a nonzero queue. In general, $t_{t,i}$ will equal the duration of the corresponding interval. However, during some intervals, the queue will decrease to 0.0 veh and $t_{t,i}$ will be only as long as the time required for the queue to dissipate (= Q_{i-1}/w_q). This condition is shown to occur during Time Interval 4 in Exhibit 31-23.

The time required for the queue to dissipate represents the queue service time. The queue can dissipate during one or more intervals for turn movements that operate in the protected-permitted mode and for shared-lane lane groups.

For lane groups with exclusive lanes and protected operation, there is one queue service time. It is followed by the green extension time.

For permitted left-turn operation in an exclusive lane, there is one queue service time. It is followed by the green extension time.

For permitted left-turn operation in a shared lane, there can be two queue service times. The green extension time follows the last service time to occur.

For protected-permitted left-turn operation in an exclusive lane, there can be two queue service times. The service time that ends during the protected period is followed by the green extension time.

For protected-permitted left-turn operation in a shared lane, there can be three queue service times. The green extension time can follow the service time that ends during the protected period, but it is more likely to follow the last service time to occur during the permitted period.

For phases serving through or right-turning vehicles in two or more lane groups, the queue service time is measured from the start of the phase to the time when the queue in each lane group has been serviced (i.e., the longest queue service time controls). This consideration is extended to lane groups with shared through and left-turning vehicles.

B. Calculate Lane Group Capacity

This step describes the procedure used to calculate lane group capacity. It is based on the QAP and considers all opportunities for service during the cycle. The equations vary, depending on the left-turn operational mode, phase sequence, and lane assignments for the subject lane group.

Protected Left-Turn Operation in Exclusive Lane

The capacity for a protected left-turn operation in an exclusive-lane lane group is computed with Equation 31-111.

Equation 31-111

$$c_{l,e,p} = \frac{g\, s_{lt}}{C} N_l$$

where $c_{l,e,p}$ is the capacity of an exclusive-lane lane group with protected left-turn operation (veh/h), N_l is the number of lanes in exclusive left-turn lane group (ln), and other variables are as previously defined.

The available capacity for the lane group is computed with Equation 31-112.

$$c_{a,l,e,p} = \frac{G_{max}\ s_{lt}}{C}\ N_l$$

Equation 31-112

where $c_{a,l,e,p}$ is the available capacity of an exclusive-lane lane group with protected left-turn operation (veh/h), G_{max} is the maximum green setting (s), and other variables are as previously defined.

Equation 31-111 and Equation 31-112 can be used to calculate the capacity of lane groups composed of through lanes and those composed of right-turn lanes with proper substitution of saturation flow rate, number of lanes, and maximum green variables.

Permitted Left-Turn Operation in Exclusive Lane

The capacity for a permitted left-turn operation in an exclusive-lane lane group is computed with Equation 31-113.

$$c_{l,e} = \frac{\min(g_p - g_f, g_u)\ s_l + 3{,}600\ n_s}{C}\ N_l$$

Equation 31-113

where $c_{l,e}$ is the capacity of an exclusive-lane lane group with permitted left-turn operation (veh/h), n_s is the number of sneakers per cycle = 2.0 (veh), and other variables are as previously defined.

The available capacity for the lane group is computed with Equation 31-114.

$$c_{a,l,e} = c_{l,e} + \frac{(G_{max} - g)\, s_l}{C}\ N_l$$

Equation 31-114

where $c_{a,l,e}$ is the available capacity of an exclusive-lane lane group with permitted left-turn operation (veh/h) and other variables are as previously defined.

In the previous equation, the saturation flow rate s_l is specifically included in the term with the maximum green setting G_{max} because this rate represents the saturation flow rate present at the end of the green interval. This is the saturation flow rate that would occur when the green is extended to its maximum green limit as a result of cycle-by-cycle fluctuations in the demand flow rate.

Permitted Left-Turn Operation in Shared Lane

The capacity for a permitted left-turn operation in a shared-lane lane group is computed with Equation 31-115.

$$c_{sl} = \frac{g_p\ s_{sl} + 3{,}600\ (1 + P_L)}{C}$$

Equation 31-115

where c_{sl} is the capacity of a shared-lane lane group with permitted left-turn operation (veh/h), s_{sl} is the saturation flow rate in shared left-turn and through lane group with permitted operation (veh/h/ln), and other variables are as previously defined.

The saturation flow rate in Equation 31-115 is computed with Equation 31-116 (all variables were previously defined).

Equation 31-116

$$s_{sl} = \frac{s_{th}}{g_p}\left(g_f + \frac{g_{diff}}{1 + P_L\left(\frac{E_{L2}}{f_{Lpb}} - 1\right)} + \frac{\min(g_p - g_f, g_u)}{1 + P_L\left(\frac{E_{L1}}{f_{Lpb}} - 1\right)} \right)$$

The available capacity for the lane group is computed with Equation 31-117.

Equation 31-117

$$c_{a,sl} = c_{sl} + \frac{(G_{max} - g_p)s_{sl3}}{C}$$

where $c_{a,sl}$ is the available capacity of a shared-lane lane group with permitted left-turn operation (veh/h) and other variables were previously defined.

In the previous equation, the saturation flow rate s_{sl3} is specifically included in the term with the maximum green setting G_{max} because this rate represents the saturation flow rate present at the end of the green interval.

Protected-Permitted Left-Turn Operation in Exclusive Lane

The capacity for a protected-permitted left-turn operation in an exclusive-lane lane group is computed with Equation 31-118.

Equation 31-118

$$c_{l,e,pp} = \left(\frac{g_l\ s_{lt}}{C} + \frac{\min(g_p - g_f, g_u)\ s_l + 3{,}600\ n_s}{C} \right) N_l$$

where $c_{l,e,pp}$ is the capacity of an exclusive-lane lane group with protected-permitted left-turn operation (veh/h) and other variables were previously defined.

The available capacity for the lane group is computed with Equation 31-119.

Equation 31-119

$$c_{a,l,e,pp} = \left(\frac{G_{max}\ s_{lt}}{C} + \frac{\min(g_p - g_f, g_u)\ s_l + 3{,}600\ n_s}{C} \right) N_l$$

where $c_{a,l,e,pp}$ is the available capacity of an exclusive-lane lane group with protected-permitted left-turn operation (veh/h) and other variables were previously defined.

In the previous equation, the saturation flow rate s_{lt} is specifically included in the term with the maximum green setting G_{max} because this rate represents the saturation flow rate present at the end of the protected green period.

Protected-Permitted Left-Turn Operation in Shared Lane

The capacity for a protected-permitted left-turn operation in a shared-lane lane group is computed with Equation 31-120.

Equation 31-120

$$c_{sl,pp} = \frac{g_l\ s_{sl4}}{C} + \frac{g_p\ s_{sl} + 3{,}600\ (1 + P_L)}{C}$$

where $c_{sl,pp}$ is the capacity of a shared-lane lane group with protected-permitted left-turn operation (veh/h) and other variables are as previously defined.

If the lane group is associated with a leading left-turn phase, then the available capacity for the lane group is computed with Equation 31-121.

$$c_{a,sl,pp} = c_{sl,pp} + \frac{\left(G_{max} - g_p\right) s_{sl3}}{C}$$

Equation 31-121

where $c_{a,sl,pp}$ is the available capacity of a shared-lane lane group with protected-permitted left-turn operation (veh/h) and other variables are as previously defined.

When the lane group is associated with a lagging left-turn phase, then the variable s_{sl3} in the previous equation is replaced by s_{sl4}.

4. QUEUE STORAGE RATIO

INTRODUCTION

This section discusses queue storage ratio as a performance measure at a signalized intersection. This measure represents the ratio of the back-of-queue size to the available vehicle storage length. The first subsection reviews concepts related to back-of-queue estimation. The second subsection describes a procedure for estimating the back-of-queue size and queue storage ratio.

The discussion in this section describes basic principles for quantifying the back of queue for selected types of lane assignment, lane grouping, left-turn operation, and phase sequence. The analyst is referred to the computational engine (see Section 7) for specific calculation details, especially as they relate to assignments, groupings, left-turn operation, and phase sequences not addressed in this section.

CONCEPTS

The *back of queue* represents the maximum backward extent of queued vehicles during a typical cycle, as measured from the stop line to the last queued vehicle. The back-of-queue size is typically reached after the onset of the green indication. The point when it is reached occurs just before the most distant queued vehicle begins forward motion as a consequence of the green indication and in response to the forward motion of the vehicle ahead.

A queued vehicle is defined to be a vehicle that is fully stopped as a consequence of the signal. A *full stop* is defined to occur when a vehicle slows to zero (or a crawl speed, if in queue) as a consequence of the change in signal indication from green to red, but not necessarily in direct response to an observed red indication.

The back-of-queue size that is estimated by the equations described here represents an overall average for the analysis period. It is represented in units of vehicles.

Background

Queue size is defined here to include only fully stopped vehicles. Vehicles that slow as they approach the back of the queue are considered to incur a *partial stop* but are not considered to be part of the queue. The distinction between a full and a partial stop is shown in Exhibit 31-24. This exhibit illustrates the trajectory of several vehicles as they traverse an intersection approach during one signal cycle. There is no residual queue at the end of the cycle.

Each thin line in Exhibit 31-24 that slopes upward from left to right represents the trajectory of one vehicle. The average time between trajectories represents the headway between vehicles (i.e., the inverse of flow rate q). The slope of the trajectory represents the vehicle's speed. The curved portion of a trajectory indicates deceleration or acceleration. The horizontal portion of a trajectory indicates a stopped condition. The effective red r and effective green g

times are shown at the top of the exhibit. The other variables shown are defined in the discussion below.

Exhibit 31-24
Time–Space Diagram of Vehicle Trajectory on an Intersection Approach

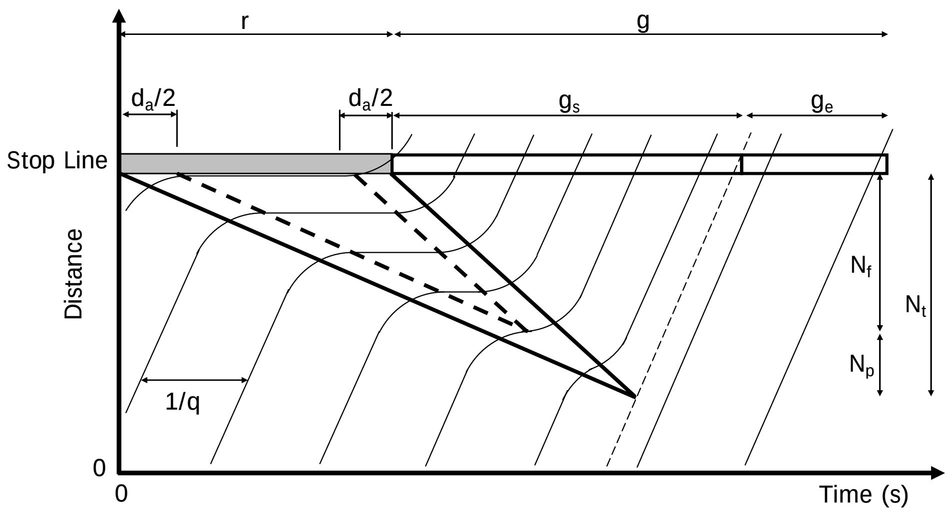


Exhibit 31-24 shows the trajectories of eight vehicles. The first five trajectories (counting from left to right) have a horizontal component to their trajectory that indicates they have reached a full stop as a result of the red indication. The sixth trajectory has some deceleration and acceleration but the vehicle does not stop. This trajectory indicates that a partial stop was incurred for the associated vehicle. The last two trajectories do not incur deceleration or acceleration, and the associated vehicles do not slow or stop. Thus, the number of full stops N_f is 5 and the number of partial stops N_p is 1. The total number of stops N_t is 6. The back-of-queue size is equal to the number of full stops.

The back-of-queue size (computed by the procedure described in the next subsection) represents the average back-of-queue for the analysis period. It is based only on those vehicles that arrive during the analysis period and join the queue. It includes the vehicles that are still in queue after the analysis period ends. The back-of-queue size for a given lane group is computed with Equation 31-122.

Equation 31-122

$$Q = Q_1 + Q_2 + Q_3$$

where

Q = back-of-queue size (veh/ln),

Q_1 = first-term back-of-queue size (veh/ln),

Q_2 = second-term back-of-queue size (veh/ln), and

Q_3 = third-term back-of-queue size (veh/ln).

The first-term back-of-queue estimate quantifies the queue size described in Exhibit 31-24. It represents the queue caused by the signal cycling through its phase sequence.

The second-term back-of-queue estimate consists of two queue components. One component accounts for the effect of random, cycle-by-cycle fluctuations in

demand that occasionally exceed capacity. This fluctuation results in the occasional overflow queue at the end of the green interval (i.e., cycle failure). The second component accounts for queuing due to a sustained oversaturation during the analysis period. This queuing occurs when aggregate demand during the analysis period exceeds aggregate capacity. It is sometimes referred to as the deterministic queue component and is shown as variable $Q_{2,d}$ in Exhibit 31-25.

Exhibit 31-25
Cumulative Arrivals and Departures During an Oversaturated Analysis Period

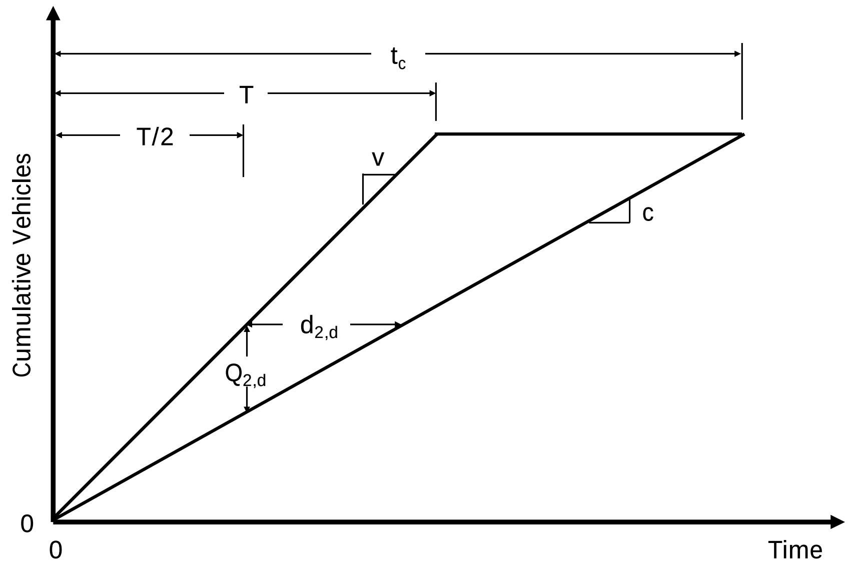


Exhibit 31-25 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate v during the analysis period T, which has capacity c. The deterministic delay component is represented by the triangular area bounded by the thick line and is associated with an average delay per vehicle represented by the variable $d_{2,d}$. The average queue size associated with this delay is shown in the exhibit as $Q_{2,d}$. The queue present at the end of the analysis period [= $T(v - c)$] is referred to as the *residual queue.*

The equation used to estimate the second-term queue is based on the assumption that no initial queue is present at the start of the analysis period. The third-term back-of-queue estimate is used to account for the additional queuing that occurs during the analysis period because of an initial queue. This queue is a result of unmet demand in the previous time period. It does *not* include any vehicles that may be in queue due to random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. When a multiple-period analysis is undertaken, the initial queue for the second and subsequent analysis periods is equal to the residual queue from the previous analysis period.

Exhibit 31-26 illustrates the queue due to an initial queue as a trapezoid shape bounded by thick lines. The average queue is represented by the variable Q_3. The initial queue size is shown as consisting of Q_b vehicles. The duration of time during the analysis period for which the effect of the initial queue is still present is represented by the variable t. This duration is shown to equal the analysis period in Exhibit 31-26. However, it can be less than the analysis period duration for some lower-volume conditions.

Exhibit 31-26
Third-Term Back-of-Queue Size with Increasing Queue

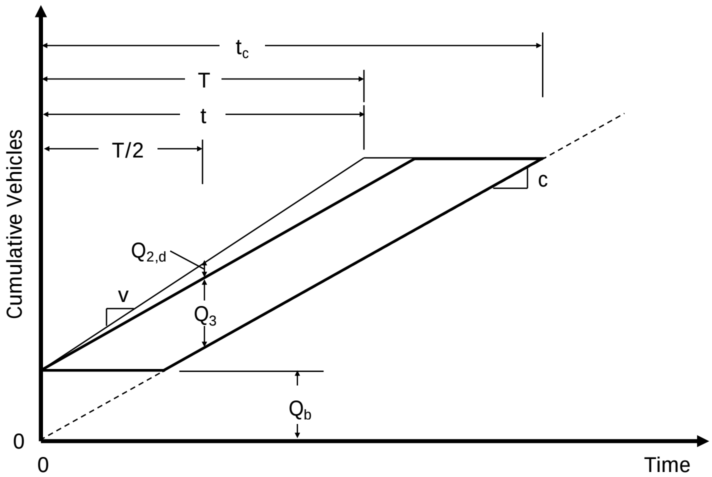


Exhibit 31-26 illustrates the case in which the demand flow rate *v* exceeds the capacity *c* during the analysis period. In contrast, Exhibit 31-27 and Exhibit 31-28 illustrate alternative cases in which the demand flow rate is less than the capacity.

Exhibit 31-27
Third-Term Back-of-Queue Size with Decreasing Queue

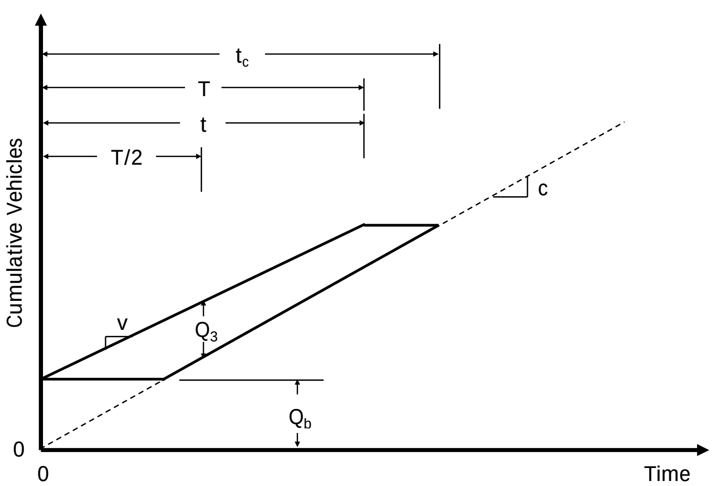
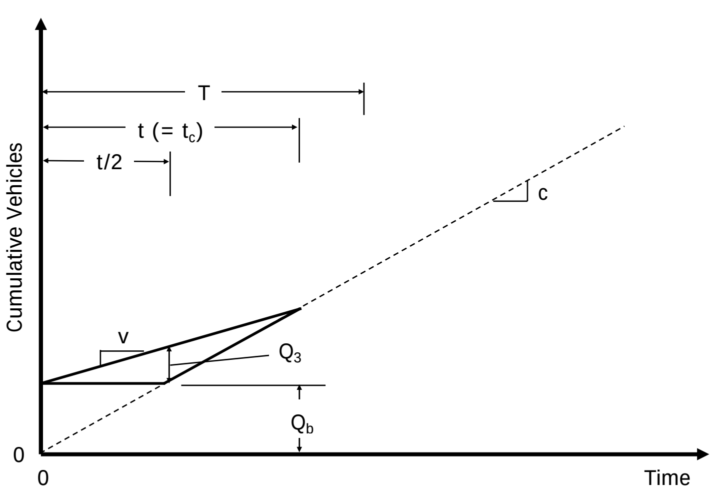


Exhibit 31-28
Third-Term Back-of-Queue Size with Queue Clearing



In this chapter, the phrase *initial queue* is always used in reference to the initial queue due to unmet demand in the previous time period. It *never* refers to vehicles in queue due to random, cycle-by-cycle fluctuations in demand.

Acceleration–Deceleration Delay

The acceleration–deceleration delay d_a term shown in Exhibit 31-24 is used to distinguish between a fully and a partially stopped vehicle. This delay term represents the time required to decelerate to a stop and then accelerate back to the initial speed, less the time it would have taken to traverse the equivalent distance at the initial speed.

Various definitions are used to describe when a vehicle is "stopped" for the purpose of field measurement. These definitions typically allow the observed vehicle to be called "stopped" even if it has a slow speed (e.g., 2 to 5 mi/h) while moving up in the queue. Many stochastic simulation programs also have a similar allowance. These practical considerations in the count of stopped vehicles require the specification of a threshold speed that can be used to identify when a vehicle is effectively stopped. The acceleration–deceleration delay for a specified threshold speed is estimated with Equation 31-123.

Equation 31-123

$$d_a = \frac{[1.47(S_a - S_s)]^2}{2\,(1.47\,S_a)}\left(\frac{1}{r_a} + \frac{1}{r_d}\right)$$

where

d_a = acceleration–deceleration delay (s),

S_a = average speed on the intersection approach (mi/h),

S_s = threshold speed defining a stopped vehicle = 5.0 (mi/h),

r_a = acceleration rate = 3.5 (ft/s^2), and

r_d = deceleration rate = 4.0 (ft/s^2).

The average speed on the intersection approach S_a is representative of vehicles that would pass unimpeded through the intersection if the signal were green for an extended period. It can be estimated with the following equation:

Equation 31-124

$$S_a = 0.90\,(25.6 + 0.47 S_{pl})$$

where S_{pl} is the posted speed limit (mi/h).

The threshold speed S_s represents the speed at or below which a vehicle is said to be effectively stopped while in queue or when joining a queue. The strictest definition of this speed is 0.0 mi/h, which coincides with a complete stop. However, vehicles sometimes move up in the queue while drivers wait for the green indication. A vehicle that moves up in the queue and then stops again does not incur an additional full stop. The threshold speed that is judged to differentiate between vehicles that truly stop and those that are just moving up in the queue is 5 mi/h.

Acceleration–deceleration delay values from Equation 31-123 typically range from 8 to 14 s, with larger values in this range corresponding to higher speeds.

Arrival–Departure Polygon

The arrival–departure polygon (ADP) associated with a lane is a graphic tool for computing the number of full stops N_f. The number of full stops has been shown to be equivalent to the first-term back-of-queue (*7*).

The ADP separately portrays the cumulative number of arrivals and departures associated with a traffic movement as a function of time during the average cycle. It is related but not identical to the QAP. The main difference is that the polygon sides in the ADP represent an arrival rate or a discharge rate but not both. In contrast, the polygon sides in the QAP represent the combined arrival and discharge rates that may occur during a common time interval.

The ADP is useful for estimating the stop rate and back-of-queue size, while the QAP is useful for estimating delay and queue service time.

The ADP for a through movement is presented in Exhibit 31-29. It shows the polygon for a typical cycle. The red and green intervals are ordered from left to right in the sequence of presentation so that the last two time periods correspond to the queue service time g_s and green extension time g_e of the subject phase. The variables shown in the exhibit are defined in the following list:

t_f = service time for fully stopped vehicles (s),

N_f = number of fully stopped vehicles (veh/ln),

g_s = queue service time (s),

g_e = green extension time (s),

q_r = arrival flow rate during the effective red time = $(1 - P)qC/r$ (veh/s),

P = proportion of vehicles arriving during the green indication (decimal),

q = arrival flow rate = $v/3{,}600$ (veh/s),

v = demand flow rate (veh/h),

r = effective red time = $C - g$ (s),

g = effective green time (s),

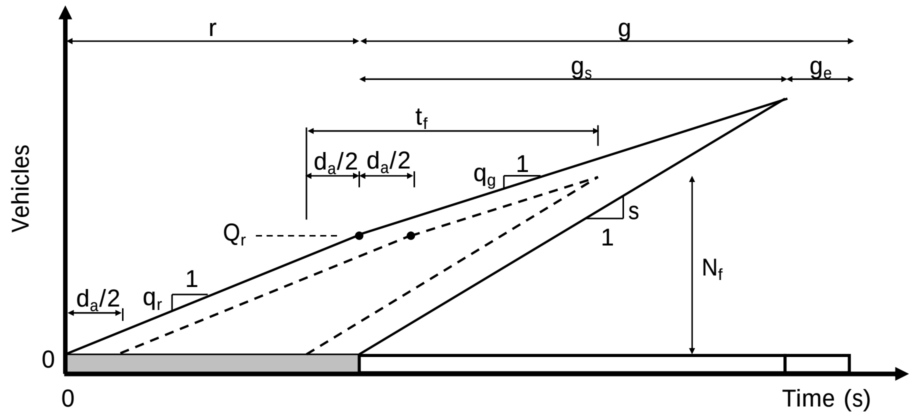
C = cycle length (s),

q_g = arrival flow rate during the effective green time = PqC/g (veh/s), and

Q_r = queue size at the end of the effective red time = $q_r r$ (veh).

In application, all flow rate variables are converted to common units of vehicles per second per lane. The presentation in this section is based on these units for q and s. If the flow rate q exceeds the lane capacity, then it is set to equal this capacity.

Exhibit 31-29
Arrival–Departure Polygon



The higher solid trend line in Exhibit 31-29 corresponds to vehicles arriving at the intersection. The lower solid trend line corresponds to queued vehicles departing the stop line. The lower trend line is horizontal during the effective red, denoting no departures. The vertical distance between these two lines at any instant in time represents the number of vehicles in the queue.

At the start of the effective red, vehicles begin to queue at a rate of q_r and accumulate to a length of Q_r vehicles at the time the effective green begins. Thereafter, the rate of arrival is q_g until the end of the effective green period. The queue service time g_s represents the time required to serve the queue present at the end of the effective red Q_r plus any additional arrivals that join the queue before it fully clears. The dashed line in this exhibit represents only those vehicles that complete a full stop. The dashed line lags behind the solid arrival line by one-half of the value of d_a (i.e., $d_a/2$). In contrast, the dashed line corresponding to initiation of the departure process leads the solid departure line by $d_a/2$.

One-half of the acceleration–deceleration delay d_a (i.e., $d_a/2$) occurs at both the end of the arrival process and the start of the discharge process. This assumption is made for convenience in developing the polygon. The derivation of the stop rate and queue length equations indicates that the two components are always combined as d_a. Thus, the assumed distribution of this delay to each of the two occurrences does not influence the accuracy of the estimated back-of-queue size.

The number of fully stopped vehicles N_f represents the number of vehicles that arrive before the queue of stopped vehicles has departed. Equation 31-125 is derived for computing this variable (all other variables previously defined).

Equation 31-125

$$N_f = q_r r + q_g \left(t_f - d_a\right)$$

Equation 31-126 can also be derived for estimating N_f.

Equation 31-126

$$N_f = \frac{s\,t_f}{3{,}600}$$

Combining Equation 31-125 and Equation 31-126 to eliminate N_f and solve for t_f yields Equation 31-127.

$$t_f = \frac{q_r r - q_g d_a}{s - q_g}$$ Equation 31-127

Equation 31-127 can be used with Equation 31-125 to obtain an estimate of N_f. The first-term back-of-queue size is then computed with the following equation:

$$Q_1 = N_f$$ Equation 31-128

The polygon in Exhibit 31-29 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. Other shapes are possible, depending on whether the lane group includes a shared lane and whether the lane group serves a permitted (or protected-permitted) left-turn movement. In general, a unique shape is dictated by each combination of left-turn operational mode (i.e., permitted, protected, protected-permitted) and phase sequence (i.e., lead, lag, split). A general procedure for constructing these polygons is described in the next subsection.

PROCEDURE FOR SELECTED LANE GROUPS

This subsection describes a procedure for estimating the back-of-queue size for a lane group at a signalized intersection. The procedure is described in a narrative format and does not define every equation needed to develop a polygon for every combination of lane allocation, left-turn operational mode, and phase sequence. This approach is taken because of the large number of equations required to address the full range of combinations found at intersections in most cities. Nevertheless, all these equations have been developed and are automated in the computational engine that is described in Section 7. Some of the equations presented in the previous section are repeated in this subsection for reader convenience.

The procedure requires the previous construction of the QAP. The construction of the QAP is described in Section 3.

Step 1. Determine Acceleration–Deceleration Delay

The acceleration–deceleration delay term is used to distinguish between fully and partially stopped vehicles. It is computed with Equation 31-129 and Equation 31-130, where all variables were previously defined.

$$d_a = \frac{[1.47(S_a - S_s)]^2}{2\,(1.47\,S_a)}\left(\frac{1}{r_a} + \frac{1}{r_d}\right)$$ Equation 31-129

with

$$S_a = 0.90\,(25.6 + 0.47 S_{pl})$$ Equation 31-130

Step 2. Define Arrival–Departure Polygon

During this step, the green times and flow rates used previously to construct the QAP are now used to construct the ADP associated with each lane group served during a phase.

The ADP in Exhibit 31-29 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. This polygon is also applicable to split phasing and to shared lane groups serving through and right-turning vehicles operating with the permitted mode. For split phasing, each approach is evaluated separately to determine its overall stop rate. If the approach has a turn lane, then a separate polygon is constructed for both the turn and the through lane groups.

More complicated combinations of phase sequence and left-turn operational mode dictate more complicated polygons. A polygon must be derived for each combination. The most common combinations are illustrated in Exhibit 31-30 to Exhibit 31-33.

The concept is extended to shared left-turn and through lane groups with protected-permitted operation in Exhibit 31-34 and Exhibit 31-35. Other polygon shapes exist, depending on traffic flow rates, phase sequence, lane use, and left-turn operational mode. The concept of construction must be extended to these other shapes to estimate accurately the back-of-queue size.

Most variables shown in these exhibits were defined in previous subsections and parts. The following variables are also defined:

g_p = effective green time for permitted left-turn operation (s),

g_u = duration of permitted left-turn green time that is not blocked by an opposing queue (s),

g_f = time before the first left-turning vehicle arrives and blocks the shared lane (s),

g_l = effective green time for left-turn phase (s),

g_{ps} = queue service time during permitted left-turn operation (s),

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln),

s_{lt} = saturation flow rate of an exclusive left-turn lane with protected operation = s_{th}/E_L (veh/h/ln),

E_L = equivalent number of through cars for a protected left-turning vehicle = 1.05,

s_{th} = saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, and area type) (veh/h/ln), and

P_L = proportion of left-turning vehicles in the shared lane (decimal).

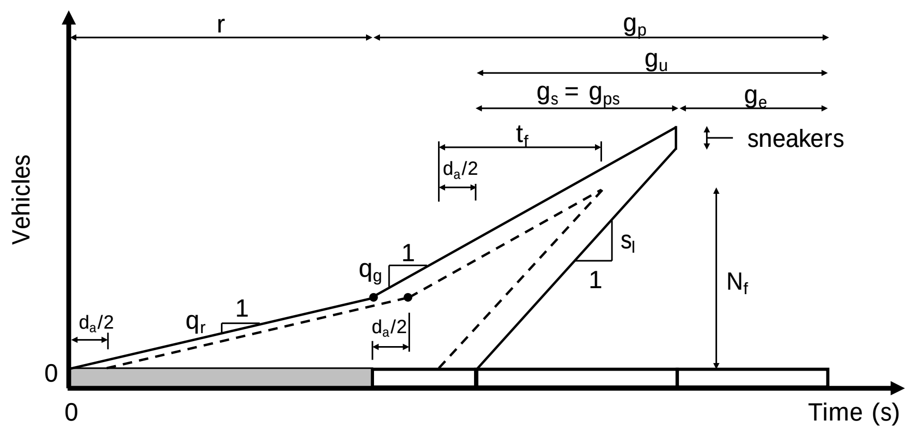


Exhibit 31-30
ADP for Permitted Left-Turn Operation in an Exclusive Lane

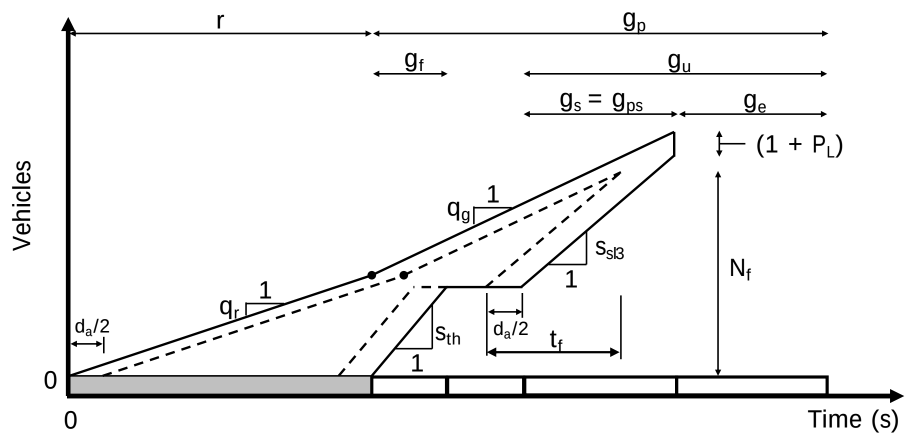


Exhibit 31-31
ADP for Permitted Left-Turn Operation in a Shared Lane

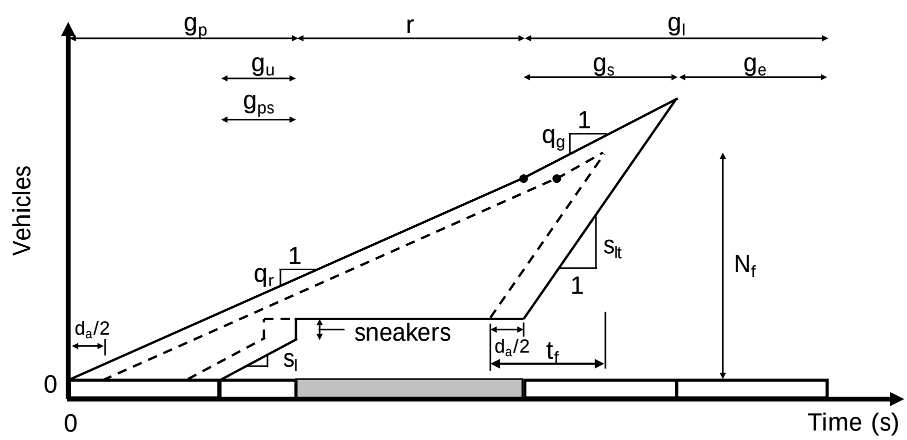


Exhibit 31-32
ADP for Leading, Protected-Permitted Left-Turn Operation in an Exclusive Lane

Exhibit 31-33
ADP for Lagging, Protected-Permitted Left-Turn Operation in an Exclusive Lane

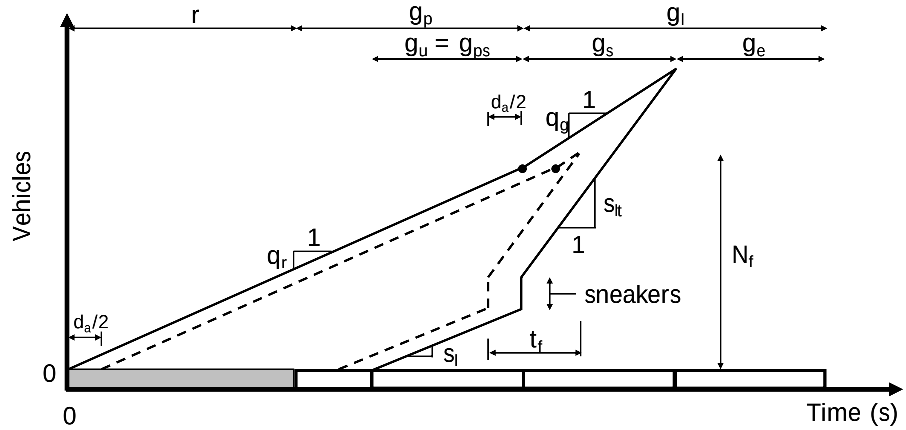


Exhibit 31-34
ADP for Leading, Protected-Permitted Left-Turn Operation in a Shared Lane

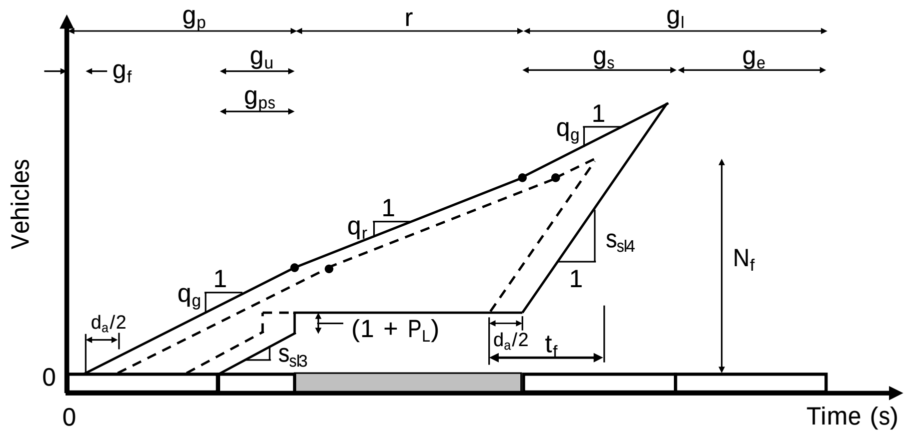
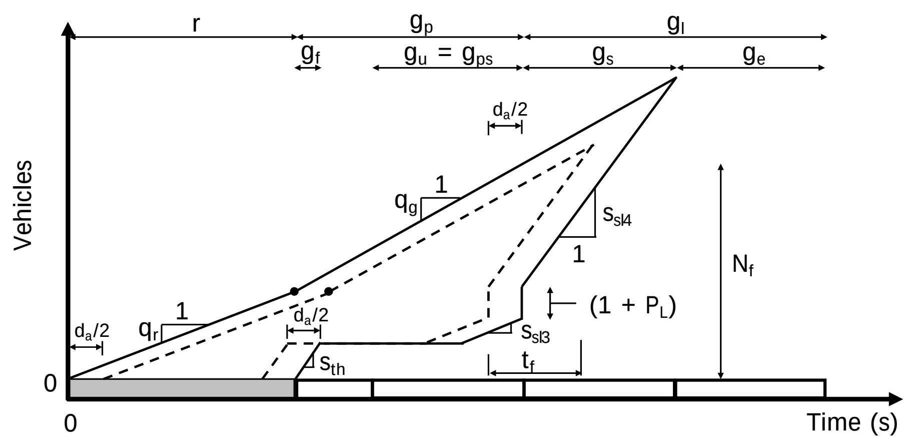


Exhibit 31-35
ADP for Lagging, Protected-Permitted Left-Turn Operation in a Shared Lane



The polygon in Exhibit 31-30 applies to the left-turn lane group served by an exclusive lane that operates in the permitted mode during the adjacent through phase. If the phase extends to max out, then some left-turning vehicles will be served as sneakers.

The polygon in Exhibit 31-31 applies to the left-turn and through lane group on a shared lane approach with permitted operation. If the phase extends to max out, then some left-turning vehicles will be served as sneakers. The expected

number of sneakers served is computed as (1 + P_L), where P_L is the proportion of left-turning vehicles in the shared lane.

The polygon in Exhibit 31-32 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and an exclusive left-turn lane. The polygon in Exhibit 31-33 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the number of sneakers.

The polygon in Exhibit 31-34 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and a shared left-turn-and-through lane group. The polygon in Exhibit 31-35 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the expected number of sneakers served (computed as 1 + P_L).

As noted previously, all polygons are based on the requirement that lane volume cannot exceed lane capacity for the purpose of estimating the queue service time. This requirement is met in the polygons shown because the queue size equals 0.0 veh at some point during the cycle.

Step 3. Define Arrival–Departure Polygon for Stopped Vehicles

During this step, the polygon defined in the previous step is enhanced to include the polygon shape for the fully stopped vehicles. The fully stopped vehicle polygon is defined by dashed lines in Exhibit 31-29 to Exhibit 31-35.

Two rules guide the development of this polygon feature. First, the dashed line that corresponds to arrivals at the stopped queue *lags* behind the solid arrival line by $d_a/2$ s. Second, the dashed line that corresponds to initiation of the departure process *leads* the solid departure line by $d_a/2$ s.

Step 4. Compute Service Time for Fully Stopped Vehicles

The service time t_f is computed for each polygon constructed in the previous step. When the polygon in Exhibit 31-29 applies, then either Equation 31-131 or Equation 31-132 can be used to compute this time.

If $d_a \leq (1 - P)gX$, then:

$$t_f = \frac{q\,C\,(1 - P - P\,d_a / g)}{s\,(1 - \min(1, X)\,P)}$$

Equation 31-131

otherwise:

$$t_f = \frac{q\,C\,(1 - P)\,(r - d_a)}{s\,(r - \min(1, X)\,[1 - P]\,g)}$$

Equation 31-132

where X is the volume-to-capacity ratio and other variables are as previously defined.

The saturation flow rate s used in Equation 31-131 and Equation 31-132 represents the adjusted saturation flow rate that is computed by the procedure described in Chapter 18, Signalized Intersections.

Step 5. Compute the Number of Fully Stopped Vehicles

The number of fully stopped vehicles N_f is computed for each polygon constructed in Step 3. When the polygon in Exhibit 31-29 applies, then Equation 31-133 or Equation 31-134 can be used to compute the number of stops (all variables are as previously defined).

If $d_a \leq (1 - P)gX$, then:

Equation 31-133

$$N_f = q_r r + q_g (t_f - d_a)$$

otherwise:

Equation 31-134

$$N_f = q_r\ (r - d_a + t_f)$$

Step 6. Compute the First-Term Back-of-Queue Size

The first-term back-of-queue estimate Q_1 (in vehicles per lane) is computed by using the number of fully stopped vehicles from the previous step. It is computed with Equation 31-135, where N_f is the number of fully stopped vehicles.

Equation 31-135

$$Q_1 = N_f$$

For some of the more complex ADPs that include left-turn movements operating with the permitted mode, the queue may dissipate at two or more points during the cycle. If this occurs, then $N_{f,i}$ is computed for each of the i periods between queue dissipation points. The first-term back-of-queue estimate is then equal to the largest of the $N_{f,i}$ values computed in this manner.

Step 7. Compute the Second-Term Back-of-Queue Size

Equation 31-136 is used to compute the second-term back-of-queue estimate Q_2 for lane groups served by an actuated phase.

Equation 31-136

$$Q_2 = \frac{c_A}{3{,}600\ N} d_2$$

where

Q_2 = second-term back-of-queue size (veh/ln),

c_A = average capacity (veh/h),

d_2 = incremental delay (s/veh), and

N = number of lanes in lane group (ln).

The procedure for calculating the average capacity c_A for the subject lane group is described in Chapter 18. If there is no initial queue, then the average capacity is equal to the lane group capacity c.

Step 8. Compute the Third-Term Back-of-Queue Size

The third-term back-of-queue estimate Q_3 is calculated with Equation 31-137 through Equation 31-142.

$$Q_3 = \frac{1}{N\ T}\left(t_A \frac{Q_b + Q_e - Q_{eo}}{2}\right)$$ **Equation 31-137**

with

$$Q_e = Q_b + t_A(v - c_A)$$ **Equation 31-138**

If $v \geq c_A$, then:

$$Q_{eo} = T(v - c_A)$$ **Equation 31-139**

$$t_A = T$$ **Equation 31-140**

If $v < c_A$, then:

$$Q_{eo} = 0.0 \text{ veh}$$ **Equation 31-141**

$$t_A = Q_b / (c_A - v) \leq T$$ **Equation 31-142**

where

- Q_3 = third-term back-of-queue size (veh/ln),
- t_A = adjusted duration of unmet demand in the analysis period (h),
- T = analysis period duration (h),
- Q_b = initial queue at the start of the analysis period (veh),
- Q_e = queue at the end of the analysis period (veh), and
- Q_{eo} = queue at the end of the analysis period when $v \geq c_A$ and $Q_b = 0.0$ (veh).

Other variables are as previously defined.

Step 9. Compute the Back-of-Queue Size

The average back-of-queue estimate Q for a lane group, in vehicles per lane, is computed with Equation 31-143 (all other variables previously defined).

$$Q = Q_1 + Q_2 + Q_3$$ **Equation 31-143**

If desired, a percentile back-of-queue estimate $Q_{\%}$ can be computed with Equation 31-144.

$$Q_{\%} = (Q_1 + Q_2) f_{B\%} + Q_3$$ **Equation 31-144**

with

If $v \geq c_A$, then:

$$f_{B\%} = \min\left(1.8,\ 1.0 + z\sqrt{\frac{I}{Q_1 + Q_2}} + 0.60\ z^{0.24}\left(\frac{g}{C}\right)^{0.33}\left(1 - e^{2-2X_A}\right)\right)$$ **Equation 31-145**

$$X_A = v / c_A$$ **Equation 31-146**

If $v < c_A$, then:

$$f_{B\%} = \min\left(1.8,\ 1.0 + z\sqrt{\frac{I}{Q_1 + Q_2}}\right)$$ **Equation 31-147**

where

$Q_{\%}$ = percentile back-of-queue size (veh/ln);

$f_{B\%}$ = percentile back-of-queue factor;

z = percentile parameter = 1.04 for 85th percentile queue, 1.28 for 90th percentile queue, 1.64 for 95th percentile queue;

I = upstream filtering adjustment factor; and

X_A = average volume-to-capacity ratio.

Other variables are as previously defined.

Step 10. Compute Queue Storage Ratio

If the lane group is served by a bay or lane of limited storage length, then the queue storage ratio can be computed with Equation 31-148.

Equation 31-148

$$R_Q = \frac{L_h\, Q}{L_a}$$

with

Equation 31-149

$$L_h = L_{pc}(1 - 0.01\, P_{HV}) + 0.01\, L_{HV}\, P_{HV}$$

where

R_Q = queue storage ratio,

L_a = available queue storage distance (ft/ln),

L_h = average vehicle spacing in stationary queue (ft/veh),

L_{pc} = stored passenger car lane length = 25 (ft),

L_{HV} = stored heavy-vehicle lane length = 45 (ft), and

P_{HV} = percent heavy vehicles in the corresponding movement group (%).

Average vehicle spacing is the average length between the front bumpers of two successive vehicles in a stationary queue. The available queue storage distance is equal to the turn bay (or lane) length.

The queue storage ratio is useful for quantifying the potential blockage of the available queue storage distance. If the queue storage ratio is less than 1.0, then blockage will not occur during the analysis period. Blockage will occur if the queue storage ratio is equal to or greater than 1.0.

If desired, a percentile queue storage ratio can be computed with Equation 31-150.

Equation 31-150

$$R_{Q\%} = \frac{L_h\, Q_{\%}}{L_a}$$

where $R_{Q\%}$ is the percentile queue storage ratio and other variables are as previously defined.

5. QUICK ESTIMATION METHOD

INTRODUCTION

This section describes a simplified method for determining the critical intersection volume-to-capacity ratio X_c, signal timing, and delay for a signalized intersection. This method can be used when minimal data are available for the analysis and only approximate results are desired.

X_c is the measure of effectiveness for quick estimation methods.

INPUT REQUIREMENTS

The overall data requirements are summarized in Exhibit 31-36. The input worksheet is shown in Exhibit 31-37. Some of the input requirements may be met by assumed values or default values. Other data items are site-specific and must be obtained in the field.

Exhibit 31-36
Input Data Requirements for Quick Estimation Method

Data Item	Comments
Volumes	By movement as projected.
Lanes	Left, through, or right; exclusive or shared.
Adjusted saturation flow rate	Includes all adjustments for PHF, CBD, grades, etc.
Left-turn phasing treatment (phasing plan)	Use actual treatment, if known. See discussion of phasing plan development.
Cycle length (minimum and maximum)	Use actual value, if known. May be estimated by using control delay and LOS worksheet.
Lost time	May be estimated by using control delay and LOS worksheet
Green times	Use actual values, if known. May be estimated by using control delay and LOS worksheet.
Coordination	Isolated intersection versus intersection influenced by upstream signals.
Peak hour factor	Use default value of 0.90 if not known.
Parking	On-street parking is or is not present.
Area type	Signal is or is not in CBD.

Note: PHF = peak hour factor, CBD = central business district, LOS = level of service.

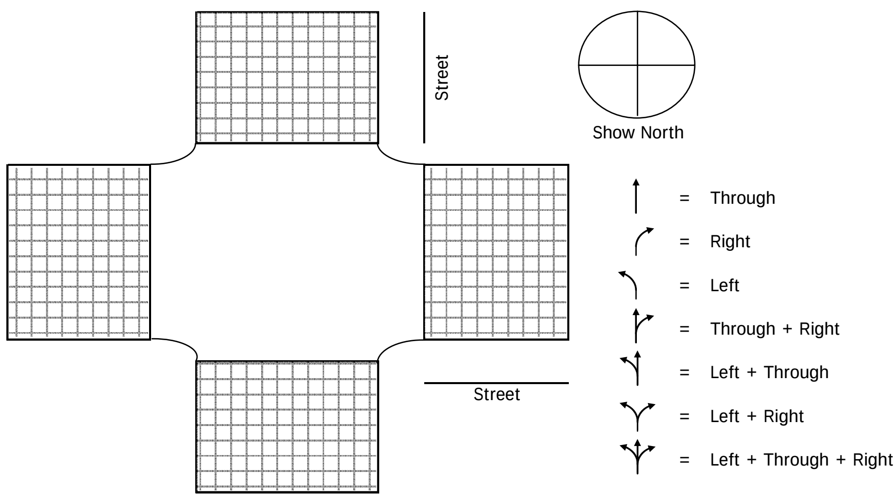
As a minimum, the analyst must provide traffic volumes and the approach lane configuration for the subject intersection. Default values for several variables are specifically identified in the methodology and integrated into the method. These values have been selected to be generally representative of typical conditions. Additional default values are identified in Section 3 of Chapter 18, Signalized Intersections.

Exhibit 31-37
Quick Estimation Input Worksheet

QUICK ESTIMATION INPUT WORKSHEET

General Information		**Site Information**	
Analyst	________	Intersection	________
Agency or Company	________	Area Type	☐ CBD ☐ Other
Date Performed	________	Jurisdiction	________
Analysis Time Period	________	Analysis Year	________

Intersection Geometry



Volume and Signal Input

	EB			WB			NB			SB		
	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]
Volume, V (veh/h)												
Proportion of LT or RT (P_{LT} or P_{RT})[2]		-			-			-			-	
Parking (Yes/No)												
Left-turn treatment (permitted, protected, not opposed) (if known)												

Peak-hour factor, PHF ________

Cycle length Minimum, C_{min} ____ s Maximum, C_{max} ____ s or Given, C ____ s

Lost time/phase ________ s

Notes

1. RT volumes, as shown, exclude RTOR.
2. P_{LT} = 1.000 for exclusive left-turn lanes, and P_{RT} = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.

METHODOLOGY

The quick estimation method consists of the five steps identified below:

1. Determine left-turn treatment,
2. Determine lane volume,
3. Determine signal timing,
4. Determine critical intersection volume-to-capacity ratio, and
5. Determine control delay.

Step 1: Determine Left-Turn Treatment

The signal timing needs of permitted left-turn movements are not considered in synthesis of the traffic signal timing plan in the quick estimation method. Therefore, failure to assume protected left-turn phases for heavy left-turn flow rates will generally produce an overly optimistic assessment of the critical volume-to-capacity ratio and intersection operations.

Exhibit 31-38 describes a procedure for determining the left-turn treatment for each intersection approach. Treatment alternatives are specific to the left-turn phase sequence and include no left-turn phase (i.e., permitted only), left-turn phase (i.e., protected), and split phasing (i.e., not opposed). The left-turn treatment checks should not be used as the sole determinant of the need for a left-turn phase.

Do not use the quick estimation method as the sole basis for determining the need for a left-turn phase.

Even if the analyst already knows that the permitted left-turn mode will be implemented, this left-turn treatment check must still be used to verify that the left-turn treatment does not conflict with the assumptions on which this quick estimation method are based. The automobile methodology presented in Chapter 18, Signalized Intersections, should be used to analyze an intersection with permitted left-turn movements that fail the left-turn treatment checks in Exhibit 31-38.

The determination of the left-turn treatment is accomplished through four checks. Once it is determined that a left-turn phase is recommended for a given intersection approach, additional checks for that approach are unnecessary.

The first check recommends use of a left-turn phase if there is more than one left-turn lane on the approach.

The second check recommends use of a left-turn phase if the unadjusted left-turn volume exceeds 240 veh/h.

The third check recommends use of a left-turn phase if the cross-product of the unadjusted left-turn and opposing mainline volumes exceeds the minimum values shown in Exhibit 31-38. The opposing mainline volume used in this step is usually the summation of the opposing through and right-turning volumes. If the opposing approach geometry is such that the subject left-turning drivers can safely ignore the opposing right-turning vehicles, then the opposing right-turn volume can be excluded from the summation. Right-turn vehicles can sometimes be ignored when there is an exclusive right-turn lane on the opposing approach and the right-turning vehicles have their own lane to turn into on the cross street (i.e., there are two or more receiving lanes on the cross street).

The fourth check compares the left-turn volume with the "sneaker" capacity and the equivalence factor (computed in the next step). The sneaker capacity represents the average number of left-turning vehicles that can complete their turn after the green interval. This check recommends the use of a left-turn phase if either the unadjusted left-turn volume exceeds the sneaker capacity or the equivalence factor exceeds 3.5.

Exhibit 31-38
Quick Estimation Left-Turn Treatment Worksheet

QUICK ESTIMATION LEFT-TURN TREATMENT WORKSHEET

General Information

Description ____________________

Check # 1. Left-Turn Lane Check

Approach	**EB**	**WB**	**NB**	**SB**
Number of left-turn lanes				
Protect left turn (Y or N)?				

If the number of left-turn lanes on any approach exceeds 1, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check # 2. Minimum Volume Check

Approach	**EB**	**WB**	**NB**	**SB**
Left-turn volume				
Protect left turn (Y or N)?				

If left-turn volume on any approach exceeds 240 veh/h, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check # 3. Minimum Cross-Product Check

Approach	**EB**	**WB**	**NB**	**SB**
Left-turn volume, V_L (veh/h)				
Opposing mainline volume, V_o (veh/h)				
Cross-product (V_L * V_o)				
Opposing through lanes				
Protected left turn (Y or N)?				

Minimum Cross-Product Values for Recommending Left-Turn Protection

Number of Through Lanes	Minimum Cross-Product
1	50,000
2	90,000
3	110,000

If the cross-product on any approach exceeds the above values, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check # 4. Sneaker Check

Approach	**EB**	**WB**	**NB**	**SB**
Left-turn volume, V_L (veh/h)				
Sneaker capacity, c_s (veh/h) c_s = 7200/C				
Equivalence factor, E_{L1}				
Protected left turn (Y or N)?				

If the equivalence factor is 3.5 or higher (computed in the Quick Estimation Lane Volume Worksheet) and the unadjusted left turn is greater than the sneaker capacity, then it is recommended that the left turns on that approach be protected.

Notes

1. If any approach is recommended for left-turn protection but the analyst evaluates it as having permitted operation, then this quick estimation method may give overly optimistic results. The analyst should instead use the methodology described in Chapter 18, Signalized Intersections.
2. All volumes used in this worksheet are unadjusted hourly volumes.

Step 2: Determine Lane Volume

The lane volume worksheet is shown in Exhibit 31-39. Its purpose is to establish the individual lane flow rate (in veh/h/ln) on each intersection approach. This information is then used in the control delay and level-of-service worksheet to synthesize the signal-timing plan. The directional designations (e.g., RT = right turn, LT = left turn) refer to the traffic movements as they approach the intersection.

Exhibit 31-39
Quick Estimation Lane Volume Worksheet

QUICK ESTIMATION LANE VOLUME WORKSHEET			
General Information			
Description/Approach			
Right-Turn Movement			
	Exclusive RT Lane		**Shared RT Lane**
RT volume, V_R (veh/h)			
Number of exclusive RT lanes, N_{RT}			use 1
RT adjustment factor,[1] f_{RT}			
RT volume per lane, V_{RT} (veh/h/ln) $V_{RT} = \frac{V_R}{(N_{RT} \times f_{RT})}$			
Left-Turn Movement			
LT volume, V_L (veh/h)			
Opposing mainline volume, V_o (veh/h)			
Number of exclusive LT lanes, N_{LT}			
LT adjustment factor,[2] f_{LT}			
LT volume per lane,[3] V_{LT} (veh/h/ln) $V_{LT} = \frac{V_L}{(N_{LT} \times f_{LT})}$	Permitted LT, use 0 ____	Protected LT ____	Not Opposed LT ____
Through Movement			
	Permitted LT	**Protected LT**	**Not Opposed LT**
Through volume, V_T (veh/h)			
Parking adjustment factor, f_p			
Number of through lanes, N_{TH}			
Total approach volume,[4] V_{tot} (veh/h) $V_{tot} = \frac{V_{RT(shared)} + V_T + V_{LT(not\ opp)}}{f_p}$			
Through Movement with Exclusive LT Lane			
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{N_{TH}}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{LT}, V_{RT} (exclusive), V_{TH}]			
Through Movement with Shared LT Lane			
Proportion of left turns, P_{LT}		Does not apply	Does not apply
Equivalence factor, E_{L1}		Does not apply	Does not apply
Shared lane LT adjustment factor, f_{DL}			use 1.0
Through volume per lane, V_{TH} (veh/h/ln) $V_{TH} = \frac{V_{tot}}{(N_{TH} \times f_{DL})}$			
Critical lane volume,[5] V_{CL} (veh/h) Max[V_{RT} (exclusive), V_{TH}]			
Notes			

1. For RT shared or single lanes, use 0.85. For RT double lanes, use 0.75.
2. For LT single lanes, use 0.95. For LT double lanes, use 0.92. For a one-way street or T-intersection, use 0.85 for one lane and 0.75 for two lanes.
3. For unopposed LT shared lanes, N_{LT} = 1.
4. For exclusive RT lanes, V_{RT} (shared) = 0. If not opposed, add V_{LT} to V_T and set $V_{LT(not\ opp)}$ = 0.
5. V_{LT} is included only if LT is unopposed. V_{RT} (exclusive) is included only if RT is exclusive.

Each of the three left-turn treatments (i.e., permitted, protected, and not opposed) must be processed differently in computing the lane volume. Therefore, the lane volume worksheet contains three columns, each representing one of the treatment alternatives. Only one of the three columns should be used for each approach. The following instructions address the procedure for completing the lane volume worksheet.

A. Compute Lane Volume for Right-Turn Movement

As a first step, remove the right-turn-on-red volume from the right-turn traffic count to obtain the right-turn volume V_R.

The right-turn adjustment factor f_{RT} is 0.85 for a single lane or a shared lane and 0.75 for two lanes.

The right-turn lane volume V_{RT} is computed by dividing the right-turn volume by the product of the number of exclusive right-turn lanes and the right-turn adjustment factor.

B. Compute Lane Volume for Left-Turn Movement

The next computation involves the left-turn volume V_L. If the left-turn movement operates in the protected-permitted mode with an exclusive left-turn lane, then two vehicles per cycle should be removed from the left-turn volume to account for the effect of sneakers. If the cycle length has not been established, then the maximum allowable cycle length should be used. To prevent unreasonably short protected left-turn phases, this volume adjustment step should not reduce the left-turn volume to a value below four vehicles per cycle.

The opposing mainline volume V_o is the total approach volume minus the left-turn volume from exclusive lanes or from a single-lane approach. The number of exclusive left-turn lanes is the number of lanes exclusively designated to accommodate the left-turn movement.

The left-turn adjustment factor f_{LT} applies to either a left-turn movement served by a left-turn phase with an exclusive left-turn lane (or lanes) or an unopposed left-turn movement. This factor is 0.95 for single lanes and 0.92 for double lanes. If the left-turn movement is not opposed because of a one-way street or T-intersection, then pedestrian interference must be considered. The corresponding value of 0.85 for one lane and 0.75 for two lanes is used.

The left-turn lane volume V_{LT} is computed by dividing the left-turn volume by the product of the number of exclusive left-turn lanes and the left-turn adjustment factor. The left-turn volume is entered directly if there is no exclusive left-turn lane. Zero should always be entered if the left turn operates in the permitted mode.

C. Compute Through-Movement Volume

The through volume V_T for the approach, excluding the left- and right-turn volumes, is placed in the appropriate row to correspond to the applicable left-turn treatment (i.e., permitted, protected, or not opposed).

The parking adjustment factor f_p is computed with Equation 31-151.

Equation 31-151

$$f_p = \frac{N_{TH} - 0.1 - \dfrac{18N_m}{3{,}600}}{N_{TH}} \geq 0.050$$

where N_m is the parking maneuver rate adjacent to lane group (maneuvers/h) and N_{TH} is the number of through lanes (shared or exclusive) (ln).

The number of through lanes N_{TH} includes any lane that serves through vehicles. Exclusive turn lanes should be excluded.

For an unopposed shared lane, the total approach volume V_{tot} is the sum of the shared-lane right-turn volume, through volume, and left-turn volume.

D. Compute Lane Volume for Through Movement with Exclusive Turn Lane

For approaches with an exclusive left-turn lane (or lanes), the through-lane volume V_{TH} is computed by dividing total approach volume by the number of through lanes.

The critical lane volume V_{CL} is normally the same as the through-lane volume, unless the right turn has an exclusive lane or the left turn is not opposed and either of these movements is more critical than the through movement. If both conditions apply, the critical lane volume will be the largest of the left-lane volume, exclusive right-lane volume, and through-lane volume.

E. Compute Lane Volume for Through Movement with Shared Lane

The computation of critical lane volume in the case of shared left-turn lanes is more complicated and requires a more detailed computational procedure. The equivalence factor E_{L1} for a permitted left turn is obtained from Exhibit 31-40 or computed with Equation 31-152.

Exhibit 31-40 Through-Car Equivalents for Permitted Left Turns

Type of Left-Turn Lane	Through-Car Equivalent E_{L1} as a Function of Opposing Flow Rate (veh/h)						
	1	200	400	600	800	1,000	1,200[a]
Shared	1.4	1.7	2.1	2.5	3.1	3.7	4.5
Exclusive	1.3	1.6	1.9	2.3	2.8	3.3	4.0

Note: [a] Use Equation 31-152, with Equation 31-153, for opposing flow in excess of 1,200 veh/h; v_o must be ≥ 0.1 veh/h.

Equation 31-152

$$E_{L1} = \frac{s_o}{s_p} - I_{sh}$$

with

Equation 31-153

$$s_p = \frac{v_o \; e^{-v_o \, t_{cg} / 3{,}600}}{1 - e^{-v_o \, t_{fh} / 3{,}600}}$$

where

E_{L1} = equivalent number of through cars for a permitted left-turning vehicle,

s_o = base saturation flow rate (pc/h/ln),

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln),

I_{sh} = indicator variable for shared lane (= 1.0 if the subject left turn is served in a shared lane, 0 if the subject left turn is served in an exclusive lane),

v_o = opposing demand flow rate (veh/h),

t_{cg} = critical headway = 4.5 (s), and

t_{fh} = follow-up headway (= 4.5 if the subject left turn is served in a shared lane, 2.5 if the subject left turn is served in an exclusive lane) (s).

An equivalence factor that exceeds 3.5 implies that left-turn capacity is derived substantially from sneakers. Therefore, if the equivalence factor is greater than 3.5 and the left-turn volume is greater than two vehicles per cycle, it is likely that the subject left turn will not have adequate capacity without a left-turn phase (with either the protected or the protected-permitted left-turn mode).

The shared-lane left-turn adjustment factor f_{DL} is computed according to Exhibit 31-41. This reduction factor is applied to the through volumes to account for the effect of left-turning vehicles waiting to turn through a gap in the opposing traffic stream. For lanes that are not opposed, this factor is 1.0 because these vehicles will have gaps through which to turn.

Exhibit 31-41
Shared-Lane Left-Turn Adjustment Factor

Permitted Left Turn

Lane groups with two or more lanes:

$$f_{DL} = \frac{(N_{TH} - 1) + e^{-(N_{TH} V_L E_{L1})/600}}{N_{TH}}$$

Subject to a minimum value that applies at very low left-turning volumes when some cycles will have no left-turn arrivals:

$$f_{DL(\min)} = \frac{(N_{TH} - 1) + e^{-(V_L C_{\max})/3{,}600}}{N_{TH}}$$

Lane groups with only one lane for all movements:

$$f_{DL} = e^{-0.02\,(E_{L1} + 10\,P_{LT})V_L\,C_{\max}\,/\,3{,}600}$$

Protected-Plus-Permitted Left Turn (one direction only)

If V_o < 1,220 veh/h, then:

$$f_{DL} = \frac{1}{1 + \left(\dfrac{P_{LT}(235 + 0.435\,V_o)}{1{,}400 - V_o}\right)}$$

If V_o ≥ 1,220 veh/h, then:

$$f_{DL} = \frac{1}{1 + 4.525\,P_{LT}}$$

The through-lane volume V_{TH} is then computed by dividing the total approach volume by the product of the number of through lanes and the shared-lane left-turn adjustment factor.

The critical lane volume V_{CL} is the maximum of either the through-lane volume or the right-turn volume from an exclusive right-turn lane. If one or more left-turn movements operate in the permitted mode, the need for a left-turn phase should be reexamined at this point.

Step 3: Determine Signal Timing

The purpose of this step is to estimate a feasible signal timing plan for the intersection. The signal timing plan is required to estimate delay. Note that the signal timing plan estimated by the method described in this step is not necessarily the optimal timing plan.

The timing plan is estimated in the following five component steps:

1. Develop phasing plan,
2. Compute critical phase volume and lost time,
3. Compute critical sum and cycle lost time,

4. Compute cycle length, and
5. Compute green time.

The information determined during these component steps is recorded in Exhibit 31-42.

Exhibit 31-42
Quick Estimation Control Delay and Level-of-Service Worksheet

QUICK ESTIMATION CONTROL DELAY AND LOS WORKSHEET

General Information			
Description ______			
East-West Phasing Plan			
Selected plan ______	**Phase No. 1**	**Phase No. 2**	**Phase No. 3**
Movement codes			
Critical phase volume, CV (veh/h)			
Lost time/phase, t_L (s)			
North-South Phasing Plan			
Selected plan ______	**Phase No. 1**	**Phase No. 2**	**Phase No. 3**
Movement codes			
Critical phase volume, CV (veh/h)			
Lost time/phase, t_L (s)			
Intersection Status Computation			
Critical sum, CS (veh/h) $CS = \Sigma CV$			
Lost time/cycle, L (s) $L = \Sigma t_L$			
Reference sum flow rate RS (veh/h)[1]			
Cycle length, C (s) $C_{min} \le C \le C_{max}$; $C = C_{max}$ when $CS \ge RS$; $C = \dfrac{L}{1 - \dfrac{CS}{RS}}$			
Critical v/c ratio, X_c $X_c = \dfrac{CS}{1700\ PHF\ f_a \left(1 - \dfrac{L}{C}\right)}$			
Intersection status (relationship to capacity)	Under ___ Near ___ At ___ Over ___		
Green Time Calculation			
East-West Phasing	**Phase No. 1**	**Phase No. 2**	**Phase No. 3**
Green time, g (s) $g = \left[(C-L)\left(\dfrac{CV}{CS}\right) + t_L\right]$			
North-South Phasing	**Phase No. 1**	**Phase No. 2**	**Phase No. 3**
Green time, g (s) $g = \left[(C-L)\left(\dfrac{CV}{CS}\right) + t_L\right]$			

Control Delay and LOS				
	EB	**WB**	**NB**	**SB**
Lane group				
Lane group adjusted volume from Lane Volume worksheet, V (veh/h)				
Green ratio, g/C				
Lane group saturation flow rate, s (veh/h) s = RS * number of lanes in lane group				
v/c ratio, X $X = \dfrac{V/s}{g/C}$				
Lane group capacity, c (veh/h) $c = \dfrac{V}{X}$				
Progression adjustment factor, PF				
Uniform delay, d_1 (s/veh)				
Incremental delay, d_2 (s/veh)				
Control delay, $d = d_1(PF) + d_2$ (s/veh)				
Delay by approach, d_A (s/veh) $\dfrac{\Sigma(d)(V)}{\Sigma V}$				
Approach flow rate, V_A (veh/h)				
Intersection delay, d_I (s/veh) $d_I = \dfrac{\Sigma(d_A)(V_A)}{\Sigma V_A}$		Intersection LOS (Exhibit 18-4)		
Notes				
1. RS = 1530 x PHFx f_a, where f_a is area adjustment factor (= 0.90 for CBD and 1.0 for all other area types).				

A. Develop Phasing Plan

The phase plan is selected from the alternatives presented in Exhibit 31-43. If the phasing plan is not known, the selection is made on the basis of the user-

specified left-turn treatment and the dominant left-turn movements identified from the left-turn treatment worksheet.

Exhibit 31-43
Phase Plans for Quick Estimation Method

Phase Plan	Eastbound	Westbound	Northbound	Southbound
1a	Permitted	Permitted	Permitted	Permitted
1b	Permitted	Not opposed	Permitted	Not opposed
1c	Not opposed	Permitted	Not opposed	Permitted
2a	Permitted	Protected	Permitted	Protected
2b	Protected	Permitted	Protected	Permitted
3a	Protected[a]	Protected	Protected[a]	Protected
3b	Protected	Protected[a]	Protected	Protected[a]
4	Not opposed	Not opposed	Not opposed	Not opposed

Note: [a] Dominant left turn for each opposing movement.

When the phase plan has been selected, the movement codes are entered in the first two sections of the control delay and level-of-service worksheet.

B. Compute Critical Phase Volume and Lost Time

The critical phase volume *CV* is the volume for the movement that requires the longest green time during the phase. If two opposing lefts are moving during the same phase, the critical phase volume is the higher-volume left turn. The appropriate choice for critical lane volume is given in the phase plan summary shown in Exhibit 31-44, along with a code that identifies the movements that are allowed to proceed in each phase. For example, NBSBTH indicates that the northbound and southbound through movements have the right-of-way on the specified phase.

Exhibit 31-44
Phase Plan Summary for Quick Estimation Method

Phase Plan	Phase No.	Lost Time (s)	East–West		North–South	
			Movement Code	Critical Volume	Movement Code	Critical Volume
1a, 1b, 1c	1	4	EBWBTH	Max(EBTH, WBTH, WBLT)	NBSBTH	Max(NBTH, NBLT, SBTH, SBLT)
2a	1 2	4 4	WBTHLT EBWBTH	WBLT Max(WBTH-WBLT, EBTH)	SBTHLT NBSBTH	SBLT Max(SBTH-SBLT, NBTH)
2b	1 2	4 4	EBTHLT EBWBTH	EBLT Max(EBTH-EBLT, WBTH)	NBTHLT NBSBTH	NBLT Max(NBTH-NBLT, SBTH)
3a	1 2 3	4 0 4	EBWBLT EBTHLT EBWBTH	WBLT EBLT-WBLT Max(WBTH, EBTH-(EBLT-WBLT))	NBSBLT NBTHLT NBSBTH	SBLT NBLT-SBLT Max(SBTH, NBTH-(NBLT-SBLT))
3b	1 2 3	4 0 4	EBWBLT WBTHLT EBWBTH	EBLT WBLT-EBLT Max(EBTH, WBTH-(WBLT-EBLT))	NBSBLT SBTHLT NBSBTH	NBLT SBLT-NBLT Max(NBTH, SBTH-(SBLT-NBLT))
4	1 2	4 4	EBTHLT WBTHLT	Max(EBTH, EBLT) Max(WBTH, WBLT)	NBTHLT SBTHLT	Max(NBTH, NBLT) Max(SBTH, SBLT)

Exhibit 31-44 also indicates the lost time assigned to each phase l_t. This time is determined for each phase and entered in the first two sections of the control delay and level-of-service worksheet. Phase lost time is incurred during any phase in which a movement is both started and stopped.

C. Compute Critical Sum and Cycle Lost Time

When all phases have been completed, the critical sum *CS* of the critical phase volumes is entered in the third section of this worksheet.

The cycle lost time *L* represents the sum of the phase lost time for each of the critical phases. For example, if Phase Plan 1 were selected for both streets, then there would be a total of 8 s of cycle lost time (4 s for each street).

D. Compute Cycle Length

If the cycle length *C* is known, then this step is skipped. If the cycle length is unknown, then it is computed by Equation 31-154.

$$C = \frac{L}{1 - \left[\frac{CS}{RS}\right]}$$

Equation 31-154

with

$$C_{min} \le C \le C_{max}$$

$$C = C_{max} \quad \text{when } CS \ge RS$$

where

C = cycle length (s),

C_{min} = minimum cycle length (s),

C_{max} = maximum cycle length (s),

L = cycle lost time (s),

CS = critical sum (veh/h),

RS = reference sum flow rate = 1,530 × $PHF \times f_a$ (veh/h),

PHF = peak hour factor, and

f_a = adjustment factor for area type = 0.90 if central business district and 1.00 otherwise.

The reference sum *RS* of phase flow rates represents the theoretical maximum value that the intersection could accommodate at an infinite cycle length. The recommended value for the reference sum is computed as an adjusted saturation flow rate. The value of 1,530 is about 90 percent of the base saturation flow rate of 1,700 pc/h/ln. The objective is to produce a volume-to-capacity ratio of 0.90 for all critical movements.

The cycle length determined from this equation should be checked against reasonable minimum and maximum values. The cycle length must not exceed a maximum allowable value set by the local jurisdiction (such as 150 s), and it must be long enough to serve pedestrians (use 60 s if local data are not available).

E. Compute Green Time

The effective green time *g* estimated in this step is based on the principle of dividing the cycle time among the conflicting phases so that the critical movements will have the same volume-to-capacity ratio. The cycle lost time must be subtracted from the cycle length to determine the effective green time per

cycle, which must then be apportioned among all phases. The effective green time per cycle is then allocated to each critical phase in proportion to the contribution of its critical phase volume to the critical sum. The effective green time for a phase is computed with Equation 31-155.

Equation 31-155

$$g = (C - L)\frac{CV}{CS}$$

where g is the effective green time (s), CV is the critical phase flow rate (veh/h), and other variables are as previously defined.

This method for estimating green time will not necessarily minimize the overall delay at the intersection.

Step 4: Determine Critical Intersection Volume-to-Capacity Ratio

The critical intersection volume-to-capacity ratio X_c is an approximate indicator of the overall sufficiency of the intersection geometrics. The computational method involves the summation of conflicting critical lane flow rates for the intersection. The computations depend on traffic signal phasing, which in turn depends on the left-turn treatment. The critical intersection volume-to-capacity ratio is computed with Equation 31-156.

Equation 31-156

$$X_c = \frac{CS}{1{,}700\ PHF\ f_a\left(1 - \frac{L}{C}\right)}$$

where X_c is the critical intersection volume-to-capacity ratio and other variables are as previously defined.

Although it is not appropriate to assign a level of service to the intersection on the basis of X_c, it is appropriate to evaluate the operational status of the intersection for quick estimation purposes. Exhibit 31-45 expresses the intersection status as over, at, near, or under capacity.

Exhibit 31-45
Intersection Status Criteria for Quick Estimation Method

Critical Volume-to-Capacity Ratio (X_c)	Relationship to Capacity
≤0.85	Under capacity
>0.85–0.95	Near capacity
>0.95–1.00	At capacity
>1.00	Over capacity

Step 5: Determine Control Delay

First, the lane groups are established for all approaches. Lane grouping is explained in Chapter 18, Signalized Intersections.

Lane group volumes V are computed by summing the adjusted volumes obtained from the lane volume worksheet for each lane group on each approach.

The green ratio g/C is computed by using the effective green time and cycle length values from the top portion of the control delay and level-of-service worksheet.

The lane group saturation flow rate s is equal to the reference sum times the number of lanes in the lane group.

The lane group volume-to-capacity ratio X is calculated by using adjusted lane group volume, lane group saturation flow rate, and green ratio.

Lane group capacity c is calculated by using lane group adjusted volume and lane group volume-to-capacity ratio.

The progression adjustment factor PF is selected from Exhibit 31-46. If the subject lane group is uncoordinated, then Arrival Type 3 is appropriate. If the subject lane group is coordinated, then Arrival Type 4 is appropriate.

Exhibit 31-46 Progression Adjustment Factor

Arrival Type	Progression Adjustment Factor *PF* as a Function of Green Ratio 0.2	0.3	0.4	0.5	0.6	0.7
Uncoordinated	1.00	1.00	1.00	1.00	1.00	1.00
Coordinated [a]	0.92	0.86	0.78	0.67	0.50	0.22

Note: [a] $PF = (1 - [1.33\ g/C])/(1 - g/C)$.

The uniform delay d_1 is computed with Equation 31-157.

Equation 31-157

$$d_1 = \frac{0.5\,C\,(1 - g/C)^2}{1 - \left[\min(1, X) g/C\right]}$$

where d_1 is uniform delay (s/veh), X is volume-to-capacity ratio, and other variables are as previously defined. The notation min(1, X) used in the equation indicates that the smaller of the two values (i.e., 1 and X) is used in the equation.

The incremental delay d_2 is computed with Equation 31-158.

Equation 31-158

$$d_2 = 900\,T\left[(X-1) + \sqrt{(X-1)^2 + \frac{4\,X}{c\,T}}\right]$$

where

d_2 = incremental delay (s/veh),

c = capacity (veh/h), and

T = analysis period duration (h).

Other variables are as previously defined.

The analysis period is equal to 0.25 h if a peak hour factor is used to estimate peak 15-min flow rates. If a peak hour factor is not used and the input volumes represent forecast hourly traffic demands, then the analysis period is 1.0 h.

The control delay is computed with Equation 31-159.

Equation 31-159

$$d = d_1(PF) + d_2$$

where d is control delay (s/veh), PF is the progression adjustment factor, and other variables are as previously defined.

Approach delay and intersection delay are calculated as a weighted average of the lane group delays. The weight for a lane group is based on the volume of each movement included in the group, as recorded on the input worksheet. The adjusted volumes used to compute capacity should not be used to compute approach delay or intersection delay. Exhibit 18-4 in Chapter 18 can be used to obtain a level-of-service estimate for the intersection, based on the computed intersection delay.

This procedure does not provide sufficient information for the computation of delay for a permitted left-turn movement from an exclusive turn lane. The analyst may ignore this delay in the computations or may use the more detailed methodology provided in Chapter 18. In addition, the quick estimation method does not include the delay associated with sneakers. The sneaker volume was subtracted from the left-turn volume associated with left-turn movements operating in a permitted mode or a protected-permitted mode.

6. FIELD MEASUREMENT TECHNIQUES

This section describes two techniques for estimating key traffic characteristics by using field data. The first subsection describes a technique for estimating control delay. The second subsection describes a technique for estimating saturation flow rate.

FIELD MEASUREMENT OF INTERSECTION CONTROL DELAY

General Notes

Delay can be measured at existing intersections as an alternative to the estimation of delay by using the methodology in Chapter 18, Signalized Intersections. There are a number of techniques for measuring delay, including a test-car survey, vehicle path tracing, input–output analysis, and queue counting. The first three techniques tend to require more time to implement than the last technique but provide more accurate delay estimates. They are often limited to sampling when implemented manually. They may be more appropriate when oversaturated conditions are present. The first two techniques can be used to estimate delay on either a movement basis or a lane group basis. The last two techniques are more amenable to delay measurement on a lane group basis.

The queue-count technique is recommended for control delay measurement. It is based on direct observation of vehicle-in-queue counts for a subject lane group. It normally requires two field personnel for each lane group surveyed. Also needed are (*a*) a multifunction digital watch that includes a countdown-repeat timer, with the countdown interval in seconds, and (*b*) a volume-count board with at least two tally counters. Alternatively, a laptop computer can be programmed to emit audio count markers at user-selected intervals, take volume counts, and execute real-time delay computations.

The queue-count technique is applicable to all undersaturated lane groups. Significant queue buildup can make the technique impractical for oversaturated lane groups or lane groups with limited storage length. If queues are lengthy, then the technique should be modified by subdividing the lane group into manageable segments (or zones) and assigning an observer to each zone. Each observer then counts queued vehicles in his or her assigned zone.

If queues are lengthy or the demand volume-to-capacity ratio is near 1.0, then care must be taken to continue the vehicle-in-queue count past the end of the arrival count period, as detailed in subsequent paragraphs. This extended counting period is required for consistency with the analytic delay equation used in the chapter text.

The technique does not directly measure delay during deceleration and during a portion of acceleration. These delay elements are very difficult to measure without sophisticated tracking equipment. Nevertheless, this technique has been shown to yield a reasonable estimate of control delay by application of appropriate adjustment factors (*8*, *9*). One adjustment factor accounts for sampling errors that may occur. Another factor accounts for unmeasured

acceleration–deceleration delay. This adjustment factor is a function of the number of vehicles in queue each cycle and the approach speed.

Approach Speed

Exhibit 31-47 shows a worksheet that can be used for recording observations and computing control delay for the subject lane group. Before starting the survey, observers need to estimate the average approach speed during the study period. Approach speed is the speed at which vehicles would pass unimpeded through the intersection if the signal were green for an extended period and volume was light. This speed may be obtained by driving through the intersection a few times when the signal is green and there is no queue. The approach speed is recorded at an upstream location that is least affected by the operation of the subject signalized intersection as well as that of any other signalized intersection.

Survey Period

The duration of the survey period must be clearly defined in advance so the last arriving vehicle or vehicles that stop in the period can be identified and counted until they exit the intersection. It is logical to define the survey period on the basis of the same considerations used to define an evaluation analysis period (as described in Chapter 18). A typical survey period is 15 min.

Count Interval

The survey technique is based on recording a vehicle-in-queue count at specific points in time. A count interval in the range of 10 to 20 s has been found to provide a good balance between delay estimate precision and observer capability. The actual count interval selected from this range is based on consideration of survey period duration and the type of control used at the intersection.

The count interval *should* be an integral divisor of the survey period duration. This characteristic ensures that a complete count of events is taken for the full survey period. It also allows easier coordination of observer tasks during the field study. For example, if the study period is 15 min, the count interval can be 10, 12, 15, 18, or 20 s.

If the intersection has pretimed or coordinated-actuated control, the count interval *should not* be an integral divisor of the cycle length. This characteristic eliminates potential survey bias due to queue buildup in a cyclical pattern. For example, if the cycle length is 120 s, the count interval can be 11, 13, 14, 16, 17, 18, or 19 s.

If the intersection has actuated control, the count interval may be chosen as the most convenient value for conducting the field survey with consideration of survey period duration.

Measurement Technique

The survey should begin at the start of the red indication associated with the subject lane group and, ideally, when no initial queue is present. If the survey period does start with an initial queue present, then these queued vehicles need

to be excluded from subsequent queue counts. This requirement stems from the need to be consistent with the delay equation in Chapter 18. This equation is derived to estimate the delay to vehicles that arrive during the survey period, not before.

Exhibit 31-47
Control Delay Field Study Worksheet

INTERSECTION CONTROL DELAY WORKSHEET

General Information

Analyst ______
Agency or Company ______
Date Performed ______
Analysis Time Period ______

Site Information

Intersection ______
Area Type ☐ CBD ☐ Other
Jurisdiction ______
Analysis Year ______

Input Initial Parameters

Number of lanes, N ______
Approach speed, S_a (mi/h) ______
Survey count interval, I_s (s) ______
Total vehicles arriving, V_{tot} ______
Stopped-vehicle count, V_{stop} ______
Cycle length, C (s) ______

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue — Count Interval 1	2	3	4	5	6	7	8	9	10
Total											

Computations

Total vehicles in queue, ΣV_{iq} = ______ veh
Time-in-queue per vehicle, $d_{vq} = \left(I_s \frac{\Sigma V_{iq}}{V_{tot}}\right) 0.9$ ______ s/veh
No. of vehicles stopping/lane/cycle = $\frac{V_{stop}}{(N_c \times N)}$ ______ veh/ln
Accel-decel correction factor, CF ______ s/veh
Number of cycles surveyed, N_c = ______
Fraction of vehicles stopping, FVS = $\frac{V_{stop}}{V_{tot}}$ ______
Accel-decel correction delay, d_{ad} = FVS * CF ______ s/veh
Control delay, d = d_{vq} + d_{ad} ______ s/veh

Observer 1 Tasks

1. Observer 1 keeps track of the end of the standing queue in each lane of the subject lane group. For purposes of the survey, a vehicle is considered as having joined the queue when it approaches within one car length of a stopped vehicle and is itself about to stop. This definition is used because of the difficulty of keeping track of the moment when a vehicle comes to a stop.

2. At the start of each count interval, Observer 1 records the number of vehicles in queue in all lanes of the subject lane group. The countdown-repeat timer on a digital watch can be used to signal the count time. This count includes vehicles that arrive when the signal is actually green but stop because queued vehicles ahead have not yet started moving. All vehicles that join a queue are included in the vehicle-in-queue count until they "exit" the intersection. A through vehicle exits the intersection when its rear axle crosses the stop line. A turning vehicle exits the intersection the instant it clears the opposing through traffic (or pedestrians to which it must yield) and begins accelerating back to the approach speed. The vehicle-in-queue count often includes some vehicles that have regained speed but have not yet exited the intersection.
3. Observer 1 records the vehicle-in-queue count in the appropriate count-interval box on the worksheet. Ten boxes are provided for each "count cycle" (note that a count cycle is not the same as a signal cycle). Any number of boxes can be used to define the count cycle; however, as many as possible should be used to ensure best use of worksheet space. The clock time at the start of the count cycle is recorded in the first column. The count cycle number is recorded in the second column of the sheet.
4. At the end of the survey period, Observer 1 continues taking vehicle-in-queue counts for all vehicles that arrived during the survey period until all of them have exited the intersection. This step requires the observer to make a mental note of the last stopping vehicle that arrived during the survey period in each lane of the lane group and continue the vehicle-in-queue counts until the last stopping vehicle or vehicles, plus all vehicles in front of the last stopping vehicles, exit the intersection. Stopping vehicles that arrive after the end of the survey period are not included in the final vehicle-in-queue counts.

Observer 2 Tasks

1. Observer 2 maintains three counts during the survey period. The first is a count of the vehicles that arrive during the survey period. The second is a count of the vehicles that arrive during the survey period and that stop one or more times. A vehicle stopping multiple times is counted only once as a stopping vehicle. The third count is the count of signal cycles, as measured by the number of times the red indication is presented for the subject lane group. For lane groups with a turn movement and protected or protected-permitted operation, the protected red indication is used for this purpose. If the survey period does not start or end at the same time as the presentation of a red indication, then the number of count intervals that occur in the interim can be used to estimate the fraction of the cycle that occurred at the start or end of the survey period.
2. Observer 2 enters all counts in the appropriate boxes on the worksheet.

Data Reduction Tasks

1. Sum each column of vehicle-in-queue counts, then sum the column totals for the entire survey period.

2. A vehicle recorded as part of a vehicle-in-queue count is assumed to be in queue, on average, for the time interval between counts. On this basis, the average time-in-queue per vehicle arriving during the survey period is estimated with Equation 31-160.

Equation 31-160

$$d_{vq} = \left(I_s \frac{\sum V_{iq}}{V_{tot}} \right) 0.9$$

where

d_{vq} = time-in-queue per vehicle (s/veh),

I_s = interval between vehicle-in-queue counts (s),

$\sum V_{iq}$ = sum of vehicle-in-queue counts (veh), and

V_{tot} = total number of vehicles arriving during the survey period (veh).

The 0.9 adjustment factor in Equation 31-160 accounts for the errors that may occur when the queue-count technique is used to estimate delay. Research has shown that the adjustment factor value is fairly constant for a variety of conditions (*8*).

3. Compute the fraction of vehicles stopping and the average number of vehicles stopping per lane in each signal cycle, as indicated on the worksheet.
4. Use Exhibit 31-48 to look up the correction factor appropriate to the lane group approach speed and the average number of vehicles stopping per lane in each cycle. This factor adjusts for deceleration and acceleration delay, which cannot be measured directly with manual techniques (*9*).

Exhibit 31-48 Acceleration–Deceleration Correction Factor

Approach Speed (mi/h)	Acceleration–Deceleration Correction Factor *CF* (s/veh) as a Function of the Average Number of Vehicles Stopping		
	≤7 veh/ln/cycle	8–19 veh/ln/cycle	20–30 veh/ln/cycle[a]
≤37	+5	+2	–1
>37–45	+7	+4	+2
>45	+9	+7	+5

Note: [a] Vehicle-in-queue counts in excess of about 30 veh/ln/cycle are typically unreliable.

5. Multiply the correction factor by the fraction of vehicles stopping. Then, add this product to the time-in-queue value from Task 2 to obtain the estimate of control delay for the subject lane group.

Example Application

Exhibit 31-49 presents sample data for a lane group during a 15-min survey period. The intersection has a 115-s cycle. A 15-s count interval is selected because it is not an integral divisor of the cycle length, but it is an integral divisor of the survey period.

Exhibit 31-49
Example Control Delay Field Study Worksheet

INTERSECTION CONTROL DELAY WORKSHEET

General Information		***Site Information***	
Analyst		Intersection	*Cicero & Belmont*
Agency or Company		Area Type	☐ CBD ☒ Other
Date Performed		Jurisdiction	
Analysis Time Period	*PM*	Analysis Year	*1999*

Input Initial Parameters			
Number of lanes, N	*2*	Total vehicles arriving, V_{tot}	*530*
Approach speed, S_a (mi/h)	*40*	Stopped-vehicle count, V_{stop}	*223*
Survey count interval, I_s (s)	*15*	Cycle length, C (s)	

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue – Count Interval 1	2	3	4	5	6	7	8	9	10
4:34	*1*	*3*	*8*	*11*	*15*	*12*	*2*	*0*	*2*		
	2	*6*	*12*	*15*	*16*	*6*	*0*	*0*	*2*		
	3	*7*	*11*	*14*	*14*	*2*	*0*	*0*			
	4	*5*	*7*	*10*	*13*	*13*	*2*	*0*	*1*		
4:42	*5*	*4*	*6*	*10*	*12*	*3*	*0*	*0*	*1*		
	6	*5*	*7*	*9*	*13*	*4*	*0*	*0*			
	7	*3*	*6*	*8*	*12*	*12*	*0*	*0*	*0*		
4:47	*8*	*4*	*7*	*11*	*16*	*9*	*0*				
Total		*37*	*64*	*88*	*111*	*61*	*4*	*0*	*6*		

Computations					
Total vehicles in queue, ΣV_{iq} =	*371*	veh	Number of cycles surveyed, N_c =	*7.8*	
Time-in-queue per vehicle, $d_{vq} = \left(I_s \dfrac{\Sigma V_{iq}}{V_{tot}}\right) 0.9$	*9.5*	s/veh	Fraction of vehicles stopping, FVS = $\dfrac{V_{stop}}{V_{tot}}$	*0.42*	
No. of vehicles stopping/lane/cycle = $\dfrac{V_{stop}}{(N_c \times N)}$	*14*	veh/ln	Accel-decel correction delay, d_{ad} = FVS * CF	*1.7*	s/veh
Accel-decel correction factor, CF	*4*	s/veh	Control delay, d = $d_{vq} + d_{ad}$	*11.2*	s/veh

The exhibit shows that data are recorded for six, seven, or eight intervals during each count cycle. This choice is arbitrary and based solely on best use of worksheet space.

The data reduction results are shown at the bottom of the exhibit. A control delay of 11.2 s/veh is estimated for the subject lane group.

Exhibit 31-50 shows how the study would have been completed if a queue remained at the end of the 15-min survey period. Only the vehicles that arrived during the 15-min period would be counted.

Exhibit 31-50
Example Worksheet with Residual Queue at End

INTERSECTION CONTROL DELAY WORKSHEET

General Information

Analyst ____
Agency or Company ____
Date Performed ____
Analysis Time Period *PM*

Site Information

Intersection *Cicero & Belmont*
Area Type ☐ CBD ☒ Other
Jurisdiction ____
Analysis Year *1999*

Input Initial Parameters

Number of lanes, N *2*
Approach speed, S_a (mi/h) *40*
Survey count interval, I_s (s) *15*

Total vehicles arriving, V_{tot} ____
Stopped-vehicle count, V_{stop} ____
Cycle length, C (s) ____

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue — Count Interval 1	2	3	4	5	6	7	8	9	10
4:47	*8*	*4*	*7*	*11*	*16*	*9*	*0*	*Queue count in previous example*			
4:47	*8*	*4*	*4**	*4**	*4**	*0*	*First four in queue have cleared by now*				
			say 15 min. survey period ends here								
			4 - last stopping vehicles in survey period; count only until they clear.*								
Total		*37*	*61*	*81*	*99*	*52*	*4*	*0*	*6*		

Computations

Total vehicles in queue, ΣV_{iq} = ____ veh

Time-in-queue per vehicle, $d_{vq} = \left(I_s \frac{\Sigma V_{iq}}{V_{tot}}\right) 0.9$ ____ s/veh

No. of vehicles stopping/lane/cycle = $\frac{V_{stop}}{(N_c \times N)}$ ____ veh/ln

Accel-decel correction factor, CF ____ s/veh

Number of cycles surveyed, N_c = ____

Fraction of vehicles stopping, FVS = $\frac{V_{stop}}{V_{tot}}$ ____

Accel-decel correction delay, d_{ad} = FVS * CF ____ s/veh

Control delay, d = d_{vq} + d_{ad} ____ s/veh

FIELD MEASUREMENT OF SATURATION FLOW RATE

This subsection describes a technique for quantifying the base saturation flow rate for local conditions. In this manner, it provides a means of calibrating the saturation flow rate calculation procedure (provided in Chapter 18) to reflect driver behavior at a local level. The technique is based on a comparison of field-measured saturation flow rate with the calculated saturation flow rate for a common set of lane groups at intersections in a given area.

Concepts

The saturation flow rate represents the maximum rate of flow in a traffic lane, as measured at the stop line during the green indication. It is usually

achieved after 10 to 14 s of green, which corresponds to the front axle of the fourth to sixth queued passenger car crossing the stop line.

The base saturation flow rate represents the saturation flow rate for a traffic lane that is 12 ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. It is usually stable over a period of time in a given area and normally exhibits a relatively narrow distribution among intersections in that area.

The prevailing saturation flow rate is the rate measured in the field for a specific lane group at a specific intersection. It may vary significantly among intersections with similar lane groups because of differences in lane width, traffic composition (i.e., percent heavy vehicles), grade, parking, bus stops, lane use, and turning vehicle operation. If the intersections are located in different areas, then the prevailing saturation flow rate may also vary because of areawide differences in the base saturation flow rate.

The adjusted saturation flow rate is the rate computed by the procedure described in Chapter 18. It represents an estimate of the prevailing saturation flow rate. It can vary among intersections for the same reasons as stated previously for the prevailing saturation flow rate. Any potential bias in the estimate is minimized by local calibration of the base saturation flow rate.

The prevailing saturation flow rate and the adjusted saturation flow rate are both expressed in units of vehicles. As a result, their value reflects the traffic composition in the subject traffic lane. In contrast, the base saturation flow rate is expressed in units of passenger cars and does not reflect traffic composition.

Measurement Technique

This part describes the technique for measuring the prevailing saturation flow rate for a given traffic lane. In general, vehicles are recorded when their front axles cross the stop line. The measurement period starts at the beginning of the green interval or when the front axle of the first vehicle in the queue passes the stop line. Saturation flow rate is calculated only from the data recorded after the fourth vehicle in the queue passes the stop line.

The vehicle’s front axle, the stop line, and the time the fourth queued vehicle crosses the stop line represent three key reference points for saturation flow measurement. Other reference points on the vehicle, on the road, or in time may yield different saturation flow rates. To maintain consistency with the methodology described in Chapter 18 and to facilitate information exchange, the aforementioned three reference points must be maintained.

If the stop line is not visible or if vehicles consistently stop beyond the stop line, then an alternative reference line must be established. This reference line should be established just beyond the typical stopping position of the first queued vehicle. Vehicles should consistently stop behind this line. Observation of several cycles before the start of the study should be sufficient to identify this substitute reference line.

The following paragraphs describe the tasks associated with a single-lane saturation flow survey. A two-person field crew is recommended. However, one

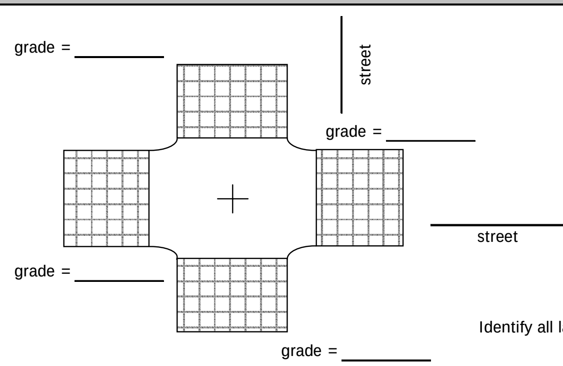
person with a tape recorder, push-button event recorder, or a notebook computer with appropriate software will suffice. The field notes and tasks identified in the following paragraphs must be adjusted according to the type of equipment used. A sample field worksheet for recording observations is included as Exhibit 31-51.

Exhibit 31-51
Saturation Flow Rate Field Study Worksheet

FIELD SATURATION FLOW RATE STUDY WORKSHEET

General Information		***Site Information***	
Analyst	______	Intersection	______
Agency or Company	______	Area Type	☐ CBD ☐ Other
Date Performed	______	Jurisdiction	______
Analysis Time Period	______	Analysis Year	______

Lane Movement Input



Movements Allowed
☐ Through
☐ Right turn
☐ Left turn

Identify all lane movements and the lane studied

Input Field Measurement

	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
Veh. in queue	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of saturation																		
End of green																		
No. veh. > 20																		
No. veh. on yellow																		

Glossary and Notes

HV = Heavy vehicles (vehicles with more than 4 tires on pavement)
T = Turning vehicles (L = Left, R = Right)
Pedestrians and buses that block vehicles should be noted with the time that they block traffic, for example,
P12 = Pedestrians blocked traffic for 12 s
B15 = Bus blocked for 15 s

General Tasks

Measure and record the area type as well as the width and grade of the lane being studied. Enter these data in the lane movement input section of the field worksheet.

Select an observation point where the roadway reference line (e.g., stop line) for the surveyed lane and the corresponding signal heads are clearly visible. When a vehicle crosses this line unimpeded, it has entered the intersection conflict space for the purpose of saturation flow measurement. Left- or right-turning vehicles yielding to opposing through traffic or yielding to pedestrians are not recorded until they proceed through the opposing traffic.

Recorder Tasks

During the measurement period, note the last vehicle in the stopped queue when the signal turns green. Describe the last vehicle to the timer. Note on the worksheet which vehicles are heavy vehicles and which vehicles turn left or right. Record the time called out by the timer.

Timer Tasks

Start the stopwatch at the beginning of the green indication and notify the recorder. Count aloud each vehicle in the queue as its front axle crosses the stop line and note the time of crossing. Call out the time of the fourth, tenth, and last vehicle in the stopped queue as its front axle is crossing the stop line.

If queued vehicles are still entering the intersection at the end of the green interval, call out (saturation through the end of green—last vehicle was number XX). Note any unusual events that may have influenced the saturation flow rate, such as buses, stalled vehicles, and unloading trucks.

The period of saturation flow begins when the front axle of the fourth vehicle in the queue crosses the roadway reference line (e.g., stop line) and ends when the front axle of the last queued vehicle crosses this line. The last queued vehicle may be a vehicle that joined the queue during the green indication.

Data Reduction

Measurements are taken cycle by cycle. To reduce the data for each cycle, the time recorded for the fourth vehicle is subtracted from the time recorded for the last vehicle in the queue. This value represents the sum of the headways for the fifth through nth vehicle, where n is the number of the last vehicle surveyed (which may not be the last vehicle in the queue). This sum is divided by the number of headways after the fourth vehicle [i.e., divided by $(n - 4)$] to obtain the average headway per vehicle under saturation flow. The saturation flow rate is 3,600 divided by this average headway.

For example, if the time for the fourth vehicle was observed as 10.2 s and the time for the 14th and last vehicle surveyed is 36.5 s, the average saturation headway per vehicle is as follows:

$$\frac{(36.5 - 10.2)}{(14 - 4)} = \frac{26.3}{10} = 2.63 \text{ s/veh}$$

The prevailing saturation flow rate in that cycle is as follows:

$$\frac{3,600}{2.63} = 1,369 \text{ veh/h/ln}$$

To obtain a statistically significant value, a minimum of 15 signal cycles (each with more than eight vehicles in the initial queue) is typically required. The average of the saturation headway per vehicle values from the individual cycles is divided into 3,600 to obtain the prevailing saturation flow rate for the surveyed lane. The percentage of heavy vehicles and turning vehicles in the sample should be determined and noted for reference.

Calibration Technique

This part describes a technique for quantifying the base saturation flow rate at a local level. It consists of three tasks. The first task entails measuring the prevailing saturation flow rate at representative locations in the local area. The second task requires the calculation of an adjusted saturation flow rate for the same locations where a prevailing saturation flow rate was measured. The third task combines the information to compute the local base saturation flow rate.

It is recognized that this technique will require some resource investment by the agency. However, it should need to be completed only once every few years. In fact, it should be repeated only when there is evidence of a change in local driver behavior. The benefit of this calibration activity will be realized by the agency in terms of more accurate estimates of automobile performance, which should translate into more effective decisions related to infrastructure investment and system management.

Task 1. Measure Prevailing Saturation Flow Rate

This task requires the measurement of prevailing saturation flow rate at one or more lane groups at each of several representative intersections in the local area. The minimum number of lane groups needed in the data set is difficult to judge for all situations; however, it should reflect a statistically valid sample. The data set should also provide reasonable geographic and physical representation of the population of signalized intersections in the local area.

The lane groups for which the prevailing saturation flow rate is measured should include a representative mix of left-turn, through, and right-turn lane groups. It should not include left-turn lane groups that operate in the permitted or the protected-permitted mode or right-turn lane groups that have protected-permitted operation. These lane groups are excluded because of the complex nature of permitted and protected-permitted operation. The saturation flow rate for these lane groups tends to have a large amount of random variation that makes it more difficult to quantify the local base saturation flow rate with an acceptable level of precision.

Once the set of lane groups is identified, the technique described in the previous part is used to measure the prevailing saturation flow rate at each location.

Task 2. Compute Adjusted Saturation Flow Rate

For this task, the saturation flow rate calculation procedure in Chapter 18 is used to compute the adjusted saturation flow rate for each lane group in the data set. If a lane group is at an intersection with actuated control for one or more phases, the automobile methodology (as opposed to just the saturation flow rate procedure) will need to be used to compute the adjusted saturation flow rate accurately. Regardless, the base saturation flow rate used with the procedure (or methodology) for this task must be 1,900 pc/h/ln.

Task 3. Compute Local Base Saturation Flow Rate

The local base saturation flow rate is computed with Equation 31-161.

Equation 31-161

$$s_{o,\text{local}} = 1{,}900\,\frac{\sum_{i=1}^{m} s_{\text{prevailing},i}}{\sum_{i=1}^{m} s_i}$$

where

$s_{o,\text{local}}$ = local base saturation flow rate (pc/h/ln),

$s_{\text{prevailing},i}$ = prevailing saturation flow rate for lane group i (veh/h/ln),

s_i = (adjusted) saturation flow rate for lane group i (veh/h/ln), and

m = number of lane groups.

Once the local base saturation flow rate $s_{o,\text{local}}$ is quantified by this technique, it is substituted thereafter for s_o in any equation in Chapter 18 that references this variable.

7. COMPUTATIONAL ENGINE DOCUMENTATION

This section uses a series of flowcharts and linkage lists to document the logic flow for the computational engine.

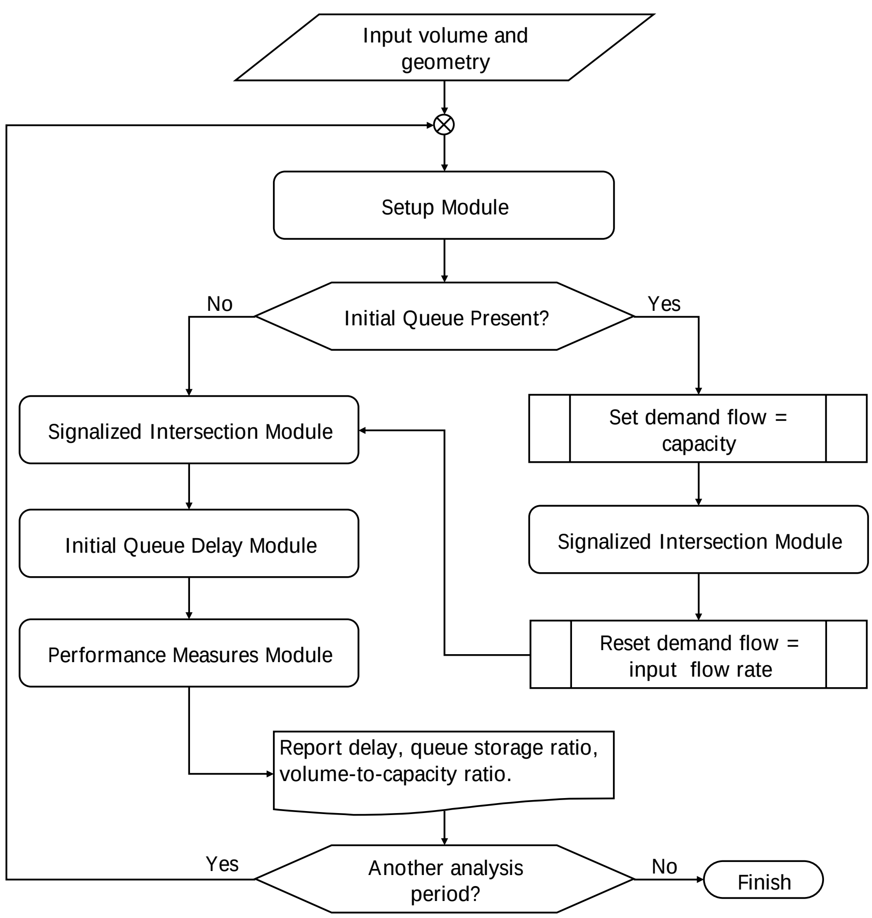
FLOWCHARTS

The methodology flowchart is shown in Exhibit 31-52. The methodology is shown to consist of four main modules:

- Setup module,
- Signalized intersection module,
- Initial queue delay module, and
- Performance measures module.

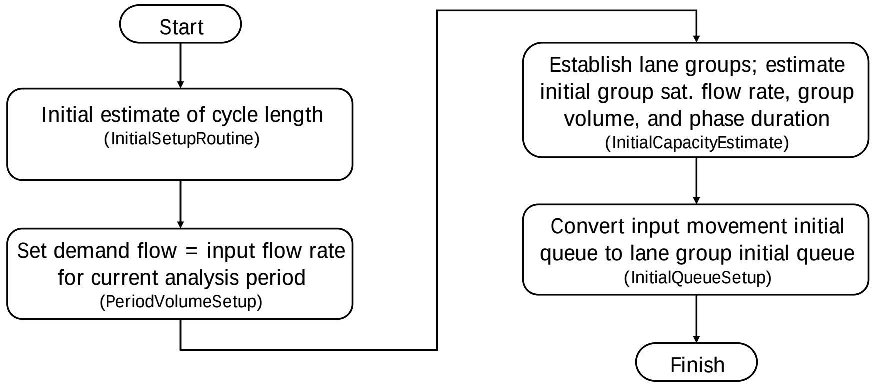
This subsection provides a separate flowchart for each of these modules.

Exhibit 31-52
Methodology Flowchart



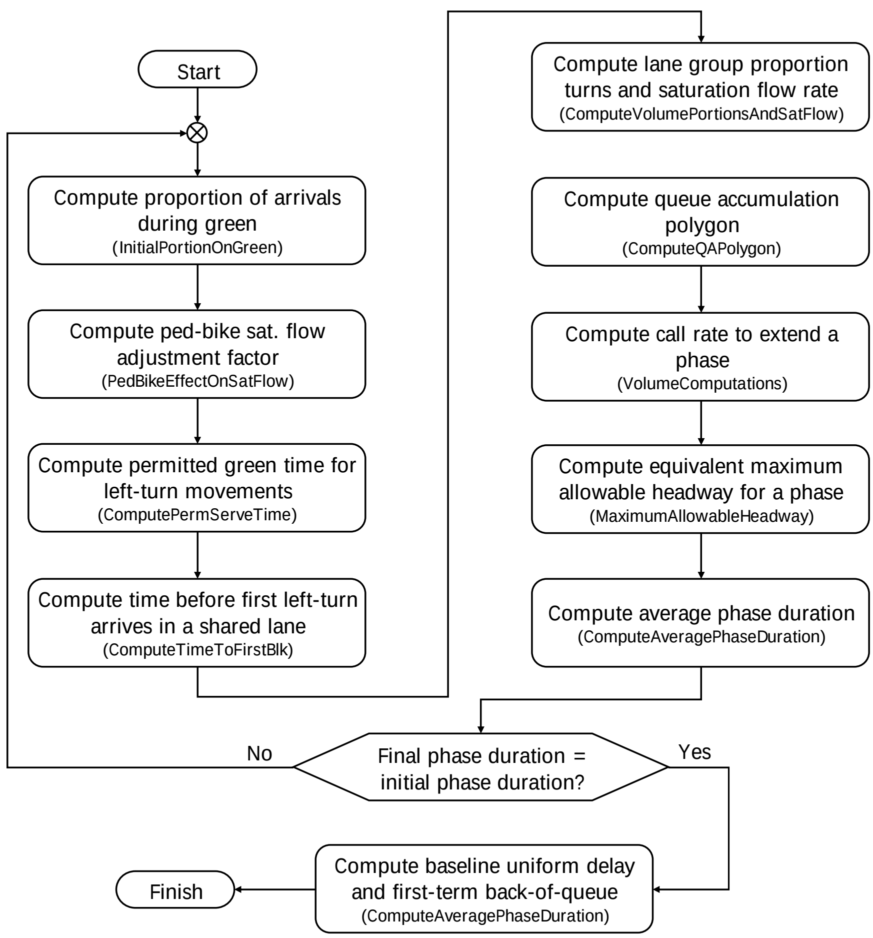
The setup module is shown in Exhibit 31-53. It consists of four main routines, as shown in the large rectangles of the exhibit. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines are described further in the next subsection.

Exhibit 31-53
Setup Module



The signalized intersection module is shown in Exhibit 31-54. It consists of nine main routines. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines are described further in the next subsection.

Exhibit 31-54
Signalized Intersection Module



The initial queue delay module is shown in Exhibit 31-55. It consists of four main routines. The main function of each routine is also shown in the exhibit.

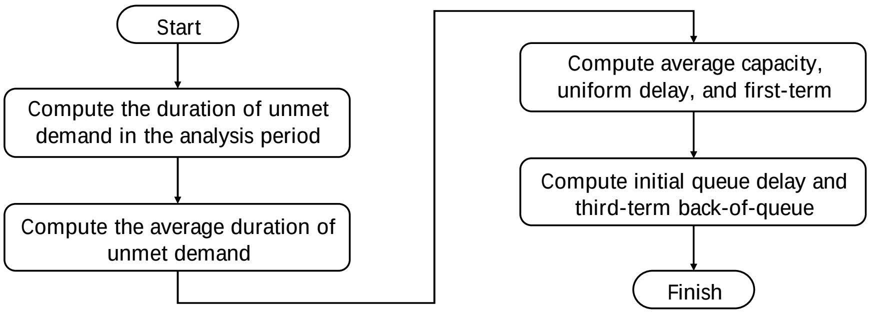


Exhibit 31-55
Initial Queue Delay Module

The performance measures module is shown in Exhibit 31-56. It consists of four main routines. The main function of each routine is also shown in the exhibit. Two of the routines are complicated enough to justify their development as separate entities in the computational engine. The name given to each of these two routines is also shown in the exhibit. These two routines are described further in the next subsection.

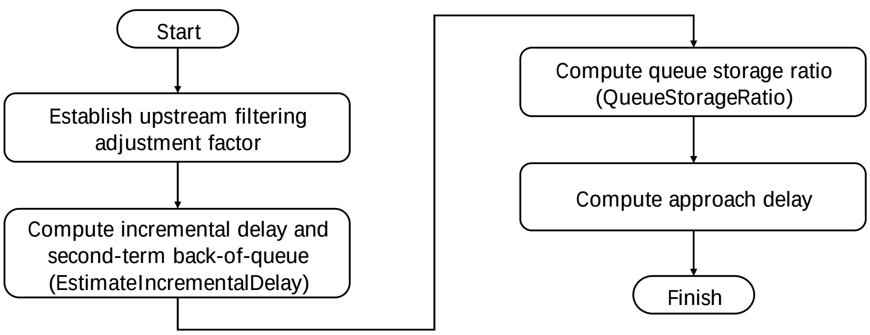


Exhibit 31-56
Performance Measures Module

LINKAGE LISTS

This subsection uses linkage lists to describe the main routines that compose the computational engine. Each list is provided in a table that identifies the routine and the various subroutines that it references. Conditions for which the subroutine is used are also provided.

The lists are organized by module, as described in the previous subsection. Five tables are provided to address the following three modules:

- Setup module (one table),
- Signalized intersection module (three tables), and
- Performance measures module (one table).

The initial queue delay module does not have a linkage list because it does not call any specific routines.

The linkage list for the setup module is provided in Exhibit 31-57. The main routines are listed in Column 1 and were previously identified in Exhibit 31-53.

Exhibit 31-57
Setup Module Routines

Routine	Subroutine	Conditions for Use
InitialSetupRoutine	Compute change period ($Y + R_c$).	None
	Compute initial estimate of cycle length C.	None
PeriodVolumeSetup	a. Compute period volume before initial queue analysis, and b. Restore period volume if initial queue analysis conducted.	Used for multiple-period analysis
	a. Save input volume as it will be overwritten if initial queue is present, and b. Restore input volume if initial queue analysis conducted.	Used for single-period analysis
InitialCapacityEstimate	getPermissiveLeftServiceTime (computes g_u, the duration of the permitted period that is not blocked by an opposing queue)	Used if subject phase serves a left-turn movement with: a. permitted mode, or b. protected-permitted mode
	getPermissiveLeftEffGreen (computes g_p, the duration of the permitted green for permitted left-turn movements)	Used if subject phase serves a left-turn movement with: a. permitted mode, or b. protected-permitted mode
	Define lane groups for each approach.	None
	Establish initial estimate of lane group volume, saturation flow rate, number of lanes capacity.	None
	Establish initial estimate of proportion turns in a shared-lane lane group.	Used for shared-lane lane groups
	PermittedSatFlow (computes permitted left-turn saturation flow rate s_p)	Used if lane group serves a left-turn movement with protected-permitted mode
	getParkBusSatFlowAdj (computes combined parking and bus blockage sat. flow adjustment factors)	Used for each lane "outside" lane group
	Establish initial estimate of queue service time g_s	None
InitialQueueSetup	Distribute input movement initial queue to corresponding lane groups.	Used for first analysis period
	Assign residual queue from last period to initial of current period, and distribute initial queue among affected lane groups.	Used for second and subsequent analysis periods

The linkage list for the signalized intersection module is provided in Exhibit 31-58 and Exhibit 31-59. The main routines are listed in Column 1 of each exhibit and were previously identified in Exhibit 31-54. The ComputeQAPolygon routine is complex enough to justify the presentation of its subroutines in a separate linkage list. This supplemental list is provided in Exhibit 31-60.

Exhibit 31-58
Signalized Intersection Module: Main Routines

Routine	Subroutine	Conditions for Use
InitialPortionOnGreen	Compute portion arriving during green *P*.	None
PedBikeEffectOnSatFlow	PedBikeEffectOnLefts	Used if subject phase serves a left-turn movement with: a. permitted mode, or b. protected-permitted mode
	PedBikeEffectOnRights	Used if subject phase serves a right-turn movement
	PedBikeEffectOnLeftsUnopposed	Used if subject phase serves a left-turn movement with split phasing
ComputePermServeTime	getPermissiveLeftServiceTime (computes g_u, the duration of the permitted period that is not blocked by an opposing queue)	Used if subject phase serves a left-turn movement with: a. permitted mode, or b. protected-permitted mode
	getPermissiveLeftEffGreen (computes g_p, the duration of the permitted green for permitted left-turn movements)	Used if subject phase serves a left-turn movement with: a. permitted mode, or b. protected-permitted mode
ComputeTimeToFirstBlk	getTimetoFirstBlk (computes g_f, the time before the first left-turning vehicle arrives and blocks the shared lane)	Used if subject phase serves a left-turn movement in a shared lane with: a. permitted mode, or b. protected-permitted mode
ComputeVolumePortions-AndSatFlow	PermittedSatFlow (computes permitted left-turn saturation flow rate s_p)	Used if lane group serves a left-turn movement with protected-permitted mode
	PortionTurnsInSharedTRlane (computes proportion of right-turning vehicles in shared lane P_R)	Used if approach has exclusive left-turn lane and subject lane group is a shared lane serving through and right-turning vehicles
	SatFlowforPermExclLefts	Used if lane group serves a left-turn movement with a permitted mode in an exclusive lane
	PortionTurnsInSharedLTRlane (computes proportion of right-turning vehicles in shared lane P_R and proportion of left-turning vehicles in shared lane P_L)	Used if approach has a shared lane serving left-turn and through vehicles
ComputeQAPolygon	QAP_ProtPermExclLane	Used if lane group serves a left-turn movement in an exclusive lane with the protected-permitted mode
	QAP_ProtMvmtExclLane	Used if lane group's movement has an exclusive lane and is serviced with protected mode
	QAP_ProtSharedLane	Used if lane group has: a. shared lane with through and right-turning movements b. a shared lane with through and left-turning movements served with split phasing
	QAP_PermLeftExclLane	Used if lane group serves a left-turn movement in an exclusive lane with the permitted mode
	QAP_PermSharedLane	Used if lane group serves a left-turn movement in a shared lane with the permitted mode

Exhibit 31-59
Signalized Intersection Module: Main Routines (continued)

Routine	Subroutine	Conditions for Use
Volume Computations	Determine call rate to extend green λ.	None
	Determine call rate to activate a phase q_v , q_p.	None
MaximumAllowable-Headway	Compute maximum allowable headway for each lane group *MAH*.	Calculations vary depending on lane group movements, lane assignment, phase sequence, and left-turn operational mode
	Compute equivalent maximum allowable headway for each phase and timer *MAH**.	None
ComputeAverage-PhaseDuration	Compute probability of green extension p.	Computed for all phases except for the timer that serves the protected left-turn movement in a shared lane
	Compute maximum queue service time for all lane groups served during the phase.	None
	Compute probability of phase termination by extension to maximum limit (i.e., max out).	None
	Compute green extension time g_e.	None
	Compute probability of a phase call p_c.	None
	Compute unbalanced green duration G_u.	None
	Compute average phase duration D_p.	None

Exhibit 31-60
Signalized Intersection Module: ComputeQAPolygon Routines

Routine	Subroutine	Conditions for Use
QAP_ProtPermExclLane	ADP_ProtPermExcl (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with left-turn movements in exclusive lane and served by protected-permitted mode
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_ProtMvmtExclLane	ADP_ProtMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with one service period
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_ProtSharedLane	ADP_ProtMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with one service period
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_PermLeftExclLane	ADP_PermLeftExclLane (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with left-turn movements in exclusive lane and served by permitted mode
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_PermSharedLane	ADP_PermSharedMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for shared-lane lane groups with a permitted left-turn movement
	ADP_ProtMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with one service period
	ADP_ProtPermShared (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with left-turn movements in shared-lane lane group and served by protected-permitted mode
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None

The linkage list for the performance measures module is provided in Exhibit 31-61. The main routines are listed in Column 1 and were previously identified in Exhibit 31-56.

Exhibit 31-61
Performance Measures Module Routines

Routine	Subroutine	Conditions for Use
EstimateIncrementalDelay	Compute incremental delay d_2 and second-term back-of-queue estimate Q_2.	None
QueueStorageRatio	Compute queue storage ratio L_Q.	None

8. SIMULATION EXAMPLES

INTRODUCTION

This section illustrates the use of alternative evaluation tools to evaluate the operation of a signalized intersection. The intersection described in Example Problem 1 from Chapter 18 is used for this purpose. There are no limitations in this example that would suggest the need for alternative tools. However, it is possible to introduce situations, such as short left-turn bays, in which an alternative tool might provide a more realistic assessment of intersection operation.

The basic intersection layout of the example intersection is shown in Exhibit 18-37. The left-turn movements on the north–south street operate under protected-permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the east–west street operate as permitted. To simplify the discussion, the pedestrian and parking activity is removed. A pretimed signal operation is used.

EFFECT OF STORAGE BAY OVERFLOW

The effect of left-turn storage bay overflow is described in this subsection as a means of illustrating the use of alternative tools. The automobile methodology in Chapter 18 can be used to compute a queue storage ratio that compares the back-of-queue estimate with the available storage length. This ratio is used to identify bays that have inadequate storage. Overflow from a storage bay can be expected to reduce approach capacity and increase the approach delay. However, these effects of bay overflow are not addressed by the automobile methodology.

Effect of Overflow on Approach Capacity and Delay

A simulation software product was selected as the alternative tool for this analysis. The intersection was simulated for a range of storage bay lengths from 0 to 250 ft. All other input data remained the same. The results presented here represent the average of 30 simulation runs for each case.

The effect of bay overflow was assessed by examining the relationship between bay length, approach throughput, and approach delay. Exhibit 31-62 shows this effect. The throughput on each approach is equal to the demand volume when storage is adequate but drops off when the bay length is decreased.

A delay comparison is also presented in Exhibit 31-62. The delay on each approach increases as bay length is reduced. The highest delay is associated with a zero-length bay, which is effectively a shared lane. The zero-length case is included here to establish a boundary condition. The delay value becomes excessive when overflow occurs. This situation often degrades into oversaturation, and a proper assessment of delay would require a multiple-period analysis to account for the buildup of long-term queues.

Exhibit 31-62
Effect of Storage Bay Length on Throughput and Delay

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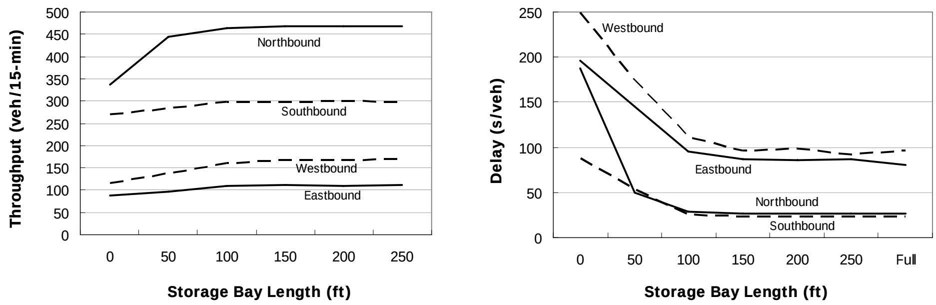

(a) Throughput (b) Delay

For case-specific applications, parameters that could influence the evaluation of bay overflow include the following:

- Number of lanes for each movement,
- Demand volumes for each movement,
- Impedance of left-turning vehicles by oncoming traffic during permitted periods,
- Signal timing plan (cycle length and phase times),
- Factors that affect the number of left-turn sneakers for left-turn movements that have permitted operation, and
- Other factors that influence the saturation flow rates.

The example intersection described here had two through lanes in all directions. If only one through lane had existed, the blockage effect would have been much more severe.

Effect of Overflow on Through Movement Capacity

This part illustrates how an alternative tool can be used to model congestion due to storage bay overflow. An example was set up involving constant blockage of a through lane by left-turning vehicles. This condition arises only under very severe oversaturation.

The following variables are used for this examination:

- Cycle length is 90 s,
- Effective green time is 41 s, and
- Saturation flow rate is 1,800 veh/h/ln.

The approach has two through lanes. Traffic volumes were sufficient to overload both lanes, so that the number of trips processed by the simulation model was determined to be an indication of through movement capacity. With no storage bay overflow effect, this capacity is computed as 1,640 veh/h (= 3,600 × 41/90). So, in a 15-min period, 410 trips were processed on average when there was no overflow.

Exhibit 31-63 shows the effect of the storage bay length on the through movement capacity. The percentage of the full capacity is plotted as a function of the storage bay length over the range of 0 to 600 ft. As expected, a zero-length bay reduces the capacity to 50% of its full value because one lane would be

constantly blocked. At the other extreme, the "no blockage" condition, achieved by setting the left-turn volume to zero, indicates that the full capacity was available. The loss of capacity is more or less linear for storage lengths up to 600 ft, at which point about 90% of the full capacity is achieved.

Exhibit 31-63
Effect of Storage Bay Length on Capacity

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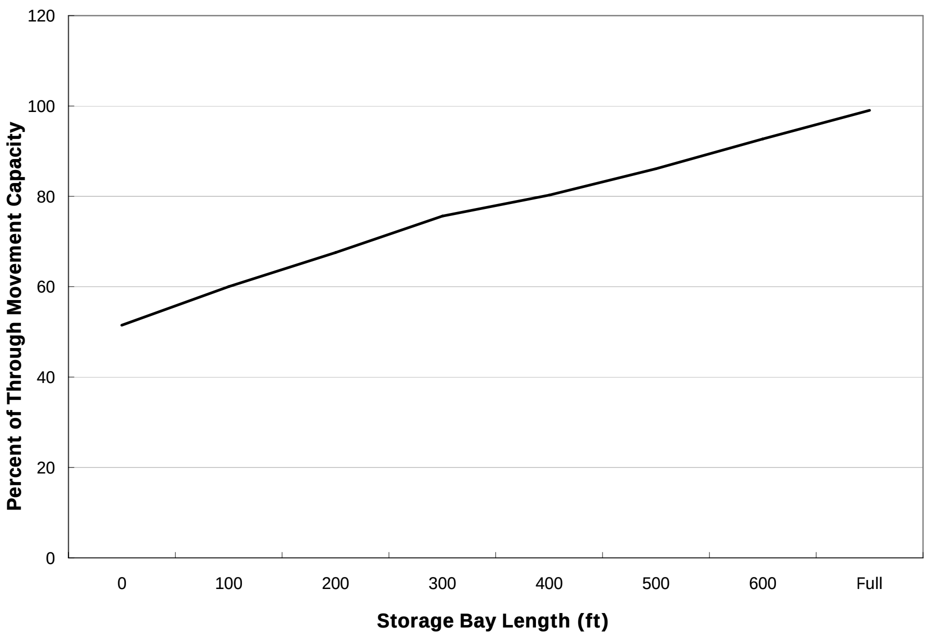


Bay overflow is a very difficult phenomenon to deal with analytically, and a substantial variation in its treatment is expected among alternative tools. The main issue for modeling is the behavior of left-turning drivers denied access to the left-turn bay because of the overflow. The animated graphics display produced by some tools can often be used to examine this behavior and assess its validity. Typically, selected model parameters can be adjusted so that the resulting behavior is more realistic.

EFFECT OF RIGHT-TURN-ON-RED OPERATION

The treatment of right-turn-on-red (RTOR) operation in the automobile methodology is limited to the removal of RTOR vehicles from the right-turn demand volume. If the right-turn movement is served by an exclusive lane, the methodology offers that the RTOR volume can be estimated as equal to the left-turn demand of the complementary cross street left-turn movement, whenever this movement is provided a left-turn phase. Given the simplicity of this treatment, it may be preferable to use an alternative tool to evaluate explicitly RTOR operation under the following conditions:

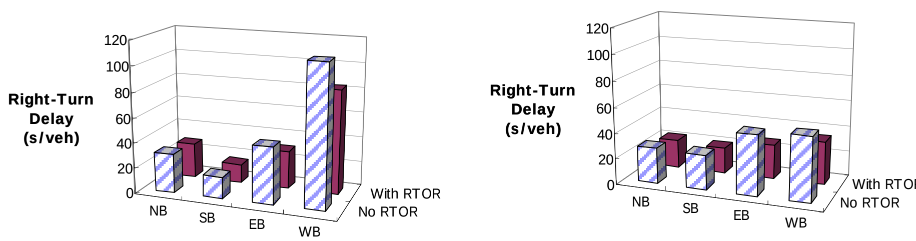
- RTOR operation occurs at the intersection,
- Right turns are a critical element of the operation,
- An acceptable level of service depends on RTOR movements, or
- Detailed phasing alternatives involving RTOR are being considered.

The remainder of this subsection examines the RTOR treatment offered in the automobile methodology. The objective of this discussion is to illustrate when alternative tools should be considered.

Effect of Right-Turn Lane Allocation

This part examines the effect of the lane allocation for the right-turn movement. The lane-allocation scenarios considered include (*a*) provision of a shared lane for the right-turn movement and (*b*) provision of an exclusive right-turn lane. Exhibit 31-64 shows the results of the analysis with the intersection in Example Problem 1 of Chapter 18. The intersection was simulated with and without the RTOR volume.

Exhibit 31-64
Effect of Right-Turn-on-Red and Lane Allocation on Delay



(a) Shared Lane (b) Exclusive Right-Turn Lane

For the most part, there are only minimal differences in delay when RTOR is allowed relative to when it is not allowed. The northbound and southbound approaches had no shadowing opportunities because the eastbound and westbound movements did not have a protected left-turn phase. As a result, the effect of lane allocation and RTOR operation was negligible for the northbound and southbound right-turn movements. In contrast, the eastbound and westbound right-turn movements were shadowed by the protected left-turn phases for the northbound and southbound approaches. As a result, the effect of lane allocation was more notable for the eastbound and the westbound right-turn movements.

Effect of Right-Turn Demand Volume

This part examines the effect of right-turn demand volume on right-turn delay, with and without RTOR allowed. The right-turn volumes varied from 100 to 400 veh/h on all approaches. Exclusive right-turn storage bays were provided on each approach.

The results are shown in Exhibit 31-65. They indicate that delay to the northbound and southbound right-turn movements was fairly insensitive to right-turn volume, with or without RTOR allowed. The available green time on these approaches provided adequate capacity for the right turns. RTOR operation provided about a 25% delay reduction.

The delay to the eastbound and westbound right-turn movements increased rapidly with right-turn volume when RTOR was not allowed. At 300 veh/h and no RTOR, the right-turn delay becomes excessive in both directions. With RTOR, delay is less sensitive to right-turn volume. This trend indicates that the additional capacity provided by RTOR is beneficial for higher right-turn volume levels.

The treatment of RTOR suggested in automobile methodology (i.e., removal of the RTOR vehicles from the right-turn volume) was also examined. The simulation analysis was repeated with the right-turn volumes reduced in this manner to explore the validity of this treatment.

Exhibit 31-65
Effect of Right-Turn-on-Red and Right-Turn Volume on Delay

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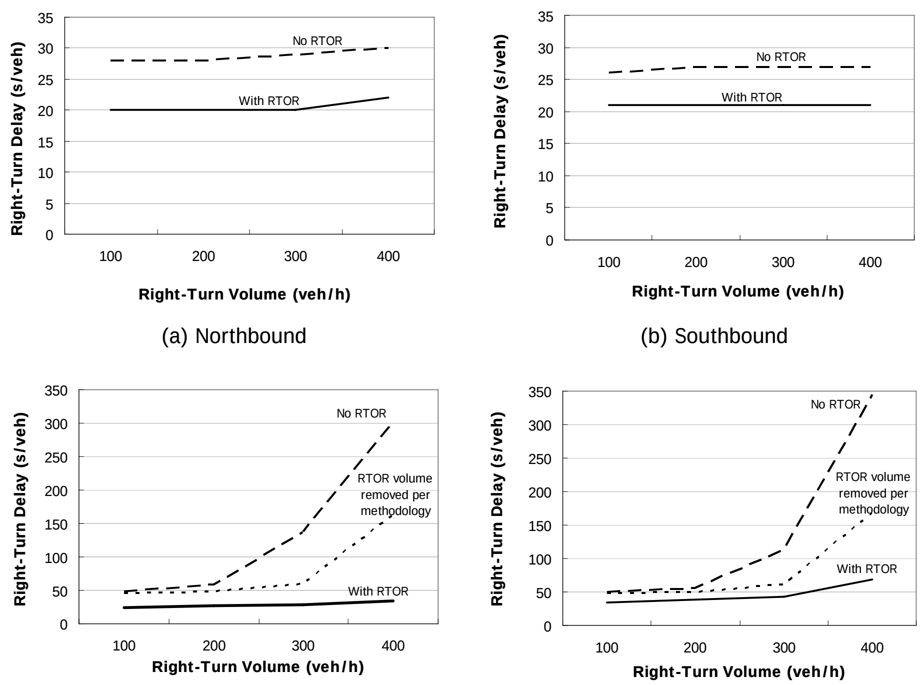

(a) Northbound (b) Southbound

(c) Eastbound (d) Westbound

The results of this analysis are shown for the eastbound and westbound approaches in Exhibit 31-65. The trends shown suggest that the treatment yields a result that is closer to the “with RTOR” case, as intended. However, use of the treatment in this case could still lead to erroneous conclusions about right-turn delay at high right-turn volumes.

Effect of a Protected Right-Turn Phase

This part examines the effect of adding a protected right-turn phase (with RTOR not allowed), relative to just allowing RTOR. The example intersection was modified to include an exclusive right-turn storage bay and a protected right-turn phase for both the eastbound and the westbound approaches. Each phase was timed concurrently with the complementary northbound or southbound left-turn phase, as appropriate. The results are shown in Exhibit 31-66. The protected phase does not improve over RTOR operation at low volume levels. However, it does provide some delay reduction at the high end of the volume scale.

Exhibit 31-66
Effect of Right-Turn-on-Red and Right-Turn Protection on Delay

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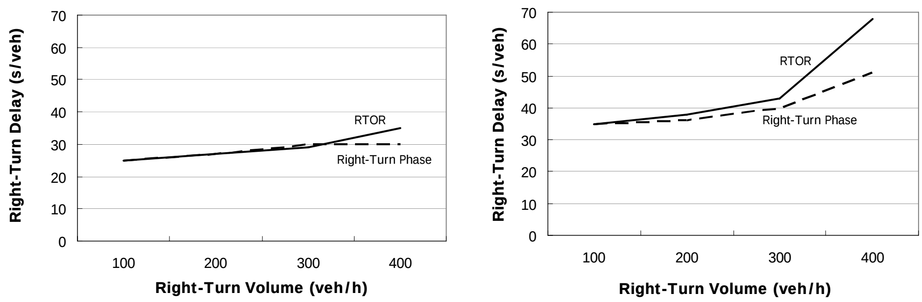


(a) Eastbound (b) Westbound

Summary

This examination indicates that RTOR operation can have some effect on right-turn delay. The effect is most notable when there are no shadowing opportunities in the phase sequence for right-turn service or the right-turn volume is high. The use of an alternative tool to evaluate RTOR operation may provide a more realistic estimate of delay than simple removal of RTOR vehicles from the right-turn demand volume, as suggested in Chapter 18.

EFFECT OF SHORT THROUGH LANES

One identified limitation of the automobile methodology is its inability to evaluate short through lanes that are added or dropped at the intersection. The intersection described in Example Problem 1 from Chapter 18 is used in this subsection to illustrate the effect of short through lanes.

Several alternative tools can address the effect of short through lanes. Each tool will have its own unique method of representing lane drop or add geometry and models of driver behavior. Some degree of approximation is involved with all evaluation tools.

The question under consideration is, “How much additional through traffic could the northbound approach accommodate if a lane were added both 150 ft upstream and 150 ft downstream of the intersection?” The capacity of the original two northbound lanes was computed as 1,778 veh/h (i.e., 889 veh/h/ln) by using the automobile methodology. Then, the simulation tool’s start-up lost time and saturation headway parameters were adjusted so that the simulation tool produced the same capacity. It was found in this case that a 2.3-s headway and 3.9-s start-up lost time produced the desired capacity.

Finally, the additional through lane was added to the simulated intersection and the process of determining capacity was repeated. On the basis of an average of 30 runs, the capacity of the additional lane was computed as 310 veh/h. Theoretically, the addition of a full lane would increase the capacity by another 889 veh/h, for a total of 2,667 veh/h.

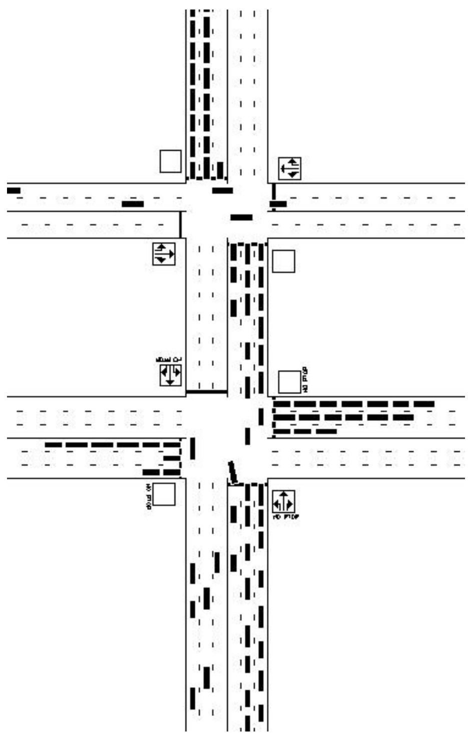
The alternative tool indicates that the additional lane contributes only 0.35 equivalent lane (= 310/889). This result cannot be stated as a general conclusion that applies to all cases because other parameters (such as the signal timing plan and the proportion of right turns in the lane group) will influence the results. More important, the results are likely to vary among alternative tools given the likely differences in their driver behavior models.

EFFECT OF CLOSELY SPACED INTERSECTIONS

The automobile methodology does not account for the effect of queue spillback from a downstream signal or demand starvation from an upstream signal. It is generally accepted that simulation of these effects is desirable when two closely spaced signalized intersections interact with each other in this manner.

The effect of closely spaced intersections is examined in this part. Consider two intersections separated by 200 ft along the north–south roadway. They operate with the same cycle length and the same northbound and southbound green time. To keep the problem simple, only through movements are allowed at these intersections. The northbound approach is used in this discussion to illustrate the effect of the adjacent intersection. The layout of this system and the resulting lane blockage are illustrated in Exhibit 31-67.

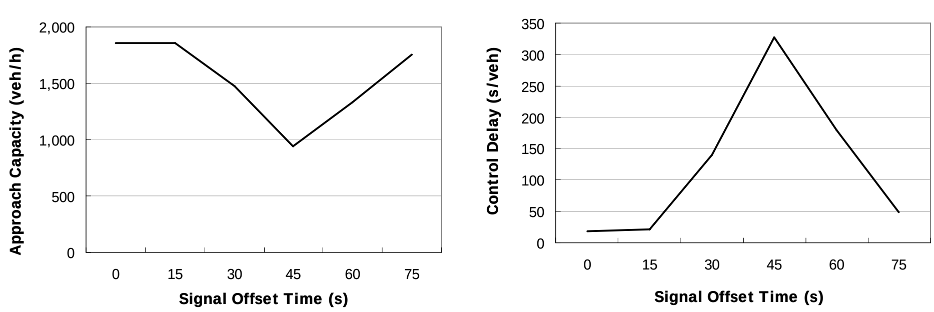
Exhibit 31-67
Closely Spaced Intersections



This exhibit illustrates both spillback and demand starvation at one point in the cycle. For the northbound direction, traffic queues have spilled back from the downstream intersection to block the upstream intersection. For the southbound direction, the traffic at the upstream intersection is prevented from reaching the downstream intersection by the red signal at the upstream intersection. Valuable green time was being wasted in both travel directions at the southern intersection.

Exhibit 31-68 illustrates the relationship between signal offset and the performance of the northbound travel direction. In terms of capacity, the exhibit shows that, under the best case condition (i.e., zero offset), the capacity is maintained at a value slightly above the demand volume. Under the worst-case conditions, the capacity is reduced to slightly below 1,000 veh/h. The demand volume-to-capacity ratio under this condition is about 1.7.

Exhibit 31-68
Effect of Closely Spaced Intersections on Capacity and Delay



(a) Approach Capacity

(b) Control Delay

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The effect of signal offset time on the delay to northbound traffic approaching the first intersection is also shown in Exhibit 31-68. As expected, the delay is minimal under favorable offsets, but it increases rapidly as the offset becomes less favorable. Delay is at its maximum value with a 45-s offset time. The large value of delay suggests that approach is severely oversaturated.

The delay reported by most simulation tools represents that incurred by vehicles when they *depart* the system during the analysis period, as opposed to the delay incurred by vehicles that *arrive* during the analysis period. The latter measure represents the delay reported by the automobile methodology.

For oversaturated conditions, the delay reported by a simulation tool may be biased when the street system is not adequately represented. This bias occurs when the street system represented to the tool does not physically extend beyond the limits of the longest queue that occurs during the analysis period.

The issues highlighted in the preceding paragraphs must be considered when an alternative tool is used. Specifically, a multiple-period analysis must be conducted that temporally spans the period of oversaturation. Also, the spatial boundaries of the street system must be large enough to encompass all queues during the saturated time periods. A more detailed discussion of multiple-period analyses is presented in Chapter 7, Interpreting HCM and Alternative Tool Results.

9. REFERENCES

Many of these references are available in the Technical Reference Library in Volume 4.

1. *National Transportation Communications for ITS Protocol: Object Definitions for Actuated Traffic Signal Controller (ASC) Units–1202*. National Electrical Manufacturers Association, Rosslyn, Va., Jan. 2005.
2. Messer, C. J., and J. A. Bonneson. *NCHRP Web Document 12: Capacity Analysis of Interchange Ramp Terminals.* Transportation Research Board, National Research Council, Washington, D.C., April 1997.
3. Zegeer, C., K. Opiela, and M. Cynecki. *Pedestrian Signalization Alternatives.* Report FHWA/RD-83/102. Federal Highway Administration, Washington, D.C., 1983.
4. Lieberman, E. B. Determining the Lateral Deployment of Traffic on an Approach to an Intersection. In *Transportation Research Record 772*, Transportation Research Board, National Research Council, Washington, D.C., 1980, pp. 1–5.
5. Bonneson, J. Lane Volume and Saturation Flow Rate for a Multilane Intersection Approach. *Journal of Transportation Engineering,* Vol. 124, No. 3, 1998, pp. 240–246.
6. Bonneson, J. A., and P. T. McCoy. *NCHRP Report 395: Capacity and Operational Effects of Midblock Left-Turn Lanes.* Transportation Research Board, National Research Council, Washington, D.C., 1997.
7. Teply, S. Accuracy of Delay Surveys at Signalized Intersections. In *Transportation Research Record 1225,* Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 24–32.
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CHAPTER 32
STOP-CONTROLLED INTERSECTIONS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. SUPPLEMENTAL MATERIAL ON TWSC POTENTIAL CAPACITY

The gap-acceptance model to estimate potential capacity (presented in Chapter 19, Equation 19-32) can be plotted for each of the non–Rank 1 movements by using values of critical headway and follow-up headway from Chapter 19 (Exhibit 19-10 and Exhibit 19-11). These graphs are presented in Exhibit 32-1, Exhibit 32-2, and Exhibit 32-3 for a major street with two lanes, four lanes, and six lanes, respectively. The potential capacity is expressed as vehicles per hour (veh/h). The exhibits indicate that the potential capacity is a function of the conflicting flow rate $v_{c,x}$, expressed as an hourly rate, as well as the type of minor-street movement.

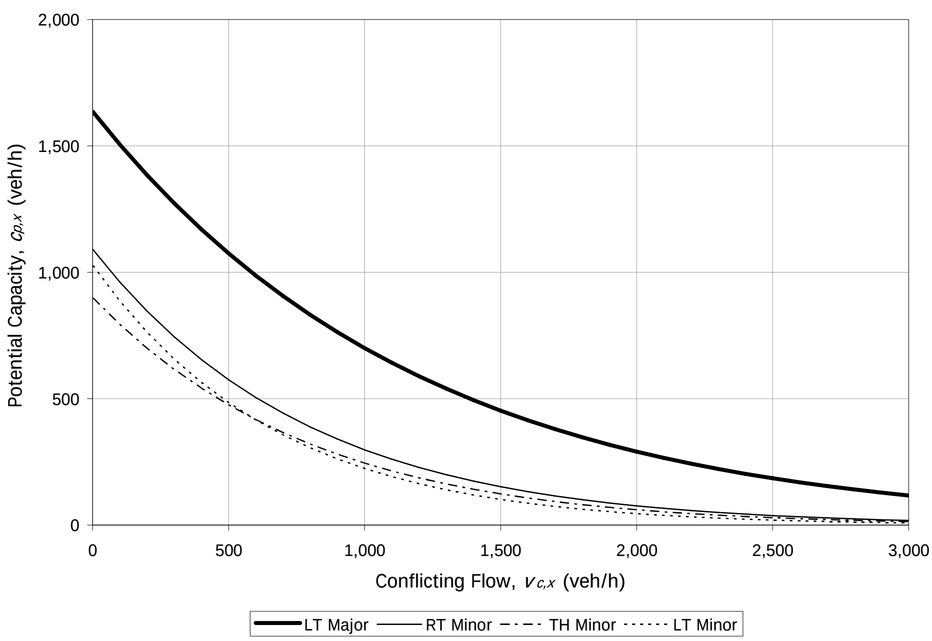

Exhibit 32-1
Potential Capacity, $c_{p,x}$, for Two-Lane Major Streets

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Exhibit 32-2
Potential Capacity, $c_{p,x}$, for Four-Lane Major Streets

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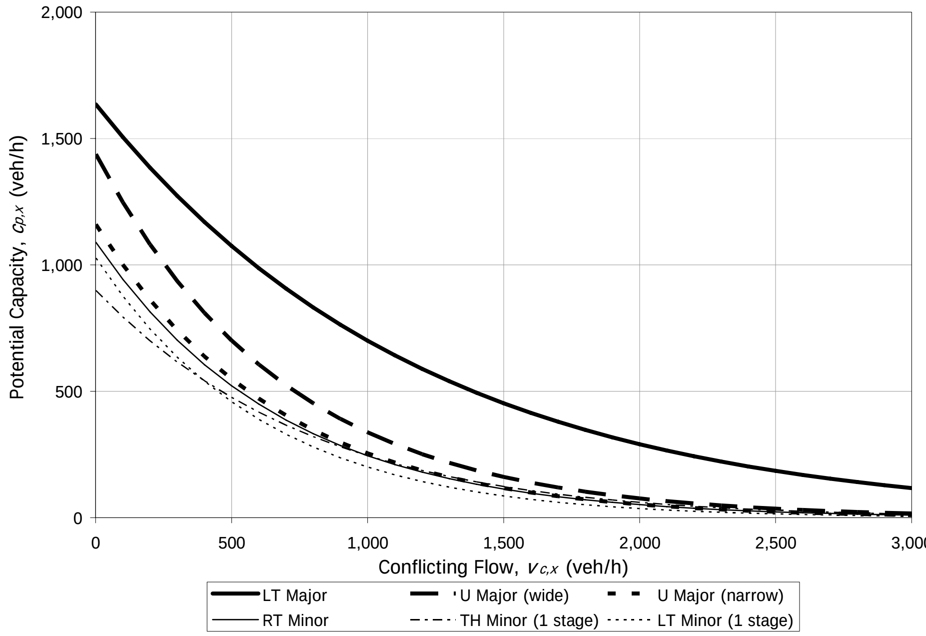

Exhibit 32-3
Potential Capacity, $c_{p,x}$, for Six-Lane Major Streets

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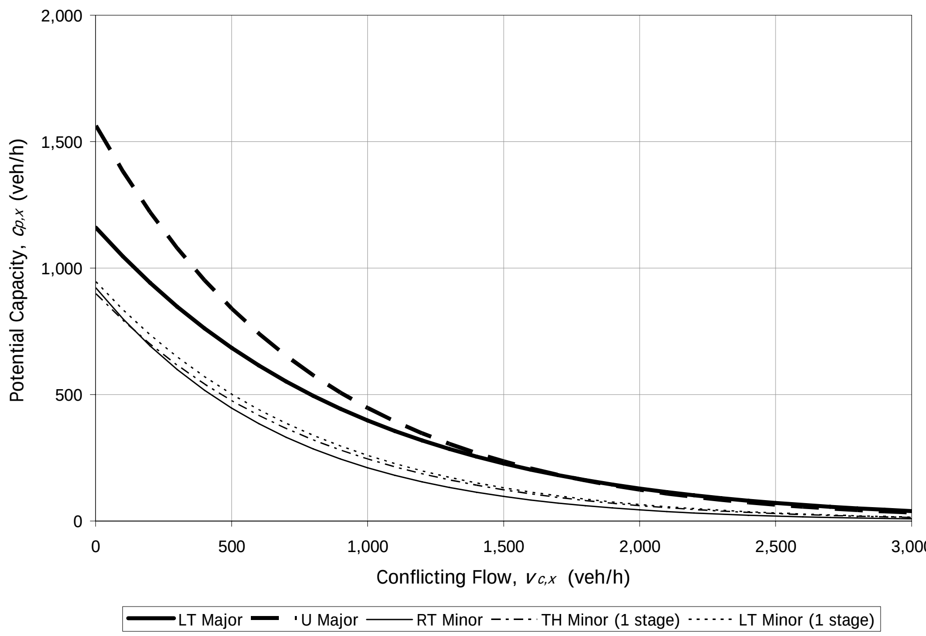


2. TWSC MOVEMENT CAPACITY ACCOUNTING FOR PEDESTRIAN EFFECTS

The following text presents the methodological details of incorporating pedestrian effects on automobile capacity. These steps replace Steps 6 through 9 presented in Chapter 19.

STEP 6: RANK 1 MOVEMENT CAPACITY

Rank 1 major-street movements are assumed to be unimpeded by any movements of lower rank. This rank also implies that major-street movements of Rank 1 are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays occasionally occur, and they are accounted for by using adjustments provided later in this procedure.

For the purposes of this procedure, major-street movements of Rank 1 are assumed to be unimpeded by pedestrians at a TWSC intersection, even though research indicates some degree of Rank 1 vehicular yielding to pedestrians (see the pedestrian methodology in Chapter 19). This is a known limitation in the procedure.

STEP 7: RANK 2 MOVEMENT CAPACITY

Movements of Rank 2 (left turns from the major street and right turns from the minor street) must yield to conflicting major-street through and right-turning vehicular movements of Rank 1 as well as conflicting pedestrian movements of Rank 1. The movement capacity of each Rank 2 movement is equal to its potential capacity, factored by any impedance due to pedestrians as indicated by Equation 32-3.

Step 7a: Pedestrian Impedance

Minor vehicular movements must yield to conflicting pedestrian movements at a TWSC intersection. A factor accounting for pedestrian blockage is computed by Equation 32-1 on the basis of pedestrian volume, pedestrian walking speed, and width of the lane the minor movement is negotiating into.

$$f_{pb} = \frac{(v_x)\left(\frac{w}{S_p}\right)}{3{,}600}$$

Equation 32-1

where

f_{pb} = pedestrian blockage factor or proportion of time that one lane on an approach is blocked during 1 h;

v_x = number of groups of pedestrians, where x is Movement 13, 14, 15, or 16;

w = width of the lane the minor movement is negotiating into (ft); and

S_p = pedestrian walking speed, assumed to be 3.5 ft/s.

The pedestrian impedance factor for pedestrian movement x, $p_{p,x}$, is computed by Equation 32-2.

Equation 32-2

$$p_{p,x} = 1 - f_{pb}$$

Exhibit 32-4 shows that Rank 2 movements v_1 and v_4 must yield to pedestrian movements v_{16} and v_{15}, respectively. Exhibit 32-4 also shows that Rank 2 movement v_9 must yield to pedestrian movements v_{15} and v_{14}, while Rank 2 movement v_{12} must yield to pedestrian movements v_{16} and v_{13}. Rank 2 U-turn movements v_{1U} and v_{4U} are assumed to not yield to pedestrians crossing the major street, consistent with the assumptions stated previously for Rank 1 vehicles.

Exhibit 32-4
Relative Pedestrian–Vehicle Hierarchy for Rank 2 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_1	v_{16}	$p_{p,16}$
v_{1U}	—	—
v_4	v_{15}	$p_{p,15}$
v_{4U}	—	—
v_9	v_{15}, v_{14}	$(p_{p,15})(p_{p,14})$
v_{12}	v_{16}, v_{13}	$(p_{p,16})(p_{p,13})$

Step 7b: Movement Capacity for Major Street Left-Turn Movements

Rank 2 major-street left-turn movements can be impeded only by conflicting pedestrians; therefore, the movement capacity for major-street left-turn movements is computed with Equation 32-3, where j denotes pedestrian movements of Rank 2 priority and i denotes movements of Rank 1 priority.

Equation 32-3

$$c_{m,j} = (c_{p,j})p_{p,i}$$

Step 7c: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity, $c_{m,j}$, for Rank 2 minor-street right-turn Movements 9 and 12 is impeded by two conflicting pedestrian movements. The capacity adjustment factors are denoted by f_9 and f_{12} for the minor-street right-turn Movements 9 and 12, respectively, and are given by Equation 32-4 and Equation 32-5, respectively.

Equation 32-4

$$f_9 = p_{p,15}p_{p,14}$$

Equation 32-5

$$f_{12} = p_{p,16}p_{p,13}$$

where

f_9, f_{12} = capacity adjustment factor for Rank 2 minor-street right-turn Movements 9 and 12, respectively; and

$p_{p,j}$ = probability that conflicting Rank 2 pedestrian movement j will operate in a queue-free state.

The movement capacity for minor-street right-turn movements is then computed with Equation 32-6:

Equation 32-6

$$c_{m,j} = (c_{p,j})f_j$$

where

$c_{m,j}$ = movement capacity for Movements 9 and 12,

$c_{p,j}$ = potential capacity for Movements 9 and 12 (from Step 5), and

f_j = capacity adjustment factor for Movements 9 and 12.

Step 7d: Movement Capacity for Major-Street U-Turn Movements

This step is the same as Step 7c in Chapter 19.

Step 7e: Effect of Major-Street Shared Through and Left-Turn Lane

This step is the same as Step 7d in Chapter 19.

STEP 8: COMPUTE MOVEMENT CAPACITIES FOR RANK 3 MOVEMENTS

Rank 3 minor-street traffic movements (minor-street through movements at four-leg intersections and minor-street left turns at three-leg intersections) must yield to conflicting Rank 1 and Rank 2 movements. Not all gaps of acceptable length that pass through the intersection are normally available for use by Rank 3 movements, because some of them are likely to be used by Rank 2 movements.

If the Rank 3 movement is a two-stage movement, the movement capacity for the one-stage movement is computed as an input to the two-stage calculation.

Step 8a: Pedestrian Impedance

Exhibit 32-5 shows that Rank 3 movements v_8 and v_{11} must yield to pedestrian movements v_{15} and v_{16}.

Exhibit 32-5
Relative Pedestrian–Vehicle Hierarchy for Rank 3 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_8	v_{15}, v_{16}	$(p_{p,15})(p_{p,16})$
v_{11}	v_{15}, v_{16}	$(p_{p,15})(p_{p,16})$

The pedestrian impedance factor for Rank 3 movements is computed according to Equation 32-1 and Equation 32-2.

Step 8b: Rank 3 Movement Capacity for One-Stage Movements

This step is the same as Step 8a in Chapter 19, except that the capacity adjustment factor, f_k, for all movements k and for all Rank 3 movements is given by Equation 32-7:

Equation 32-7

$$f_k = \prod_j (p_{0,j}) p_{p,x}$$

where

$p_{0,j}$ = probability that conflicting Rank 2 movement j will operate in a queue-free state,

$p_{p,x}$ = probability of pedestrian movements of Rank 1 or Rank 2 priority,

k = Rank 3 movements, and

x = 13, 14, 15, 16 (pedestrian movements of both Rank 1 and Rank 2).

Step 8c: Rank 3 Capacity for Two-Stage Movements

This step is the same as Step 8b in Chapter 19.

STEP 9: COMPUTE MOVEMENT CAPACITIES FOR RANK 4 MOVEMENTS

Rank 4 movements occur only at four-leg intersections. Rank 4 movements (i.e., only the minor-street left turns at a four-leg intersection) can be impeded by all higher-ranked movements (Ranks 1, 2, and 3).

Step 9a: Rank 4 Pedestrian Impedance

Exhibit 32-6 shows that Rank 4 movement v_7 must yield to pedestrian movements v_{15} and v_{13}, and Rank 4 movement v_{10} must yield to pedestrian movements v_{16} and v_{14}.

Exhibit 32-6
Relative Pedestrian–Vehicle Hierarchy for Rank 4 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_7	v_{15}, v_{13}	$(p_{p,15})(p_{p,13})$
v_{10}	v_{16}, v_{14}	$(p_{p,16})(p_{p,14})$

The pedestrian impedance factor for Rank 4 movements is computed according to Equation 32-1 and Equation 32-2.

Step 9b: Rank 4 Capacity for One-Stage Movements

This step is the same as Step 9a in Chapter 19, except that the capacity adjustment factor for the Rank 4 minor-street left-turn movement can be computed by Equation 32-8:

Equation 32-8

$$f_l = (p')(p_{0,j})(p_{p,x})$$

where

l = minor-street left-turn movement of Rank 4,

j = conflicting Rank 2 minor-street right-turn movement, and

$p_{p,x}$ = the values shown in Equation 32-2 (the variable $p_{0,j}$ should be included only if movement j is identified as a conflicting movement).

Step 9c: Rank 4 Capacity for Two-Stage Movements

This step is the same as Step 9b in Chapter 19.

3. TWSC SUPPLEMENTAL EXAMPLE PROBLEMS

This part of the chapter provides additional example problems for use of the TWSC methodology. Exhibit 32-7 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operations analysis level in terms of the calculations, except that default values are used where available.

Example Problems 1 and 2 appear in Chapter 19.

Exhibit 32-7 TWSC Example Problems

Problem Number	Description	Analysis Level
3	TWSC intersection with flared approaches and median storage	Operational
4	TWSC intersection within a signalized urban street segment	Operational
5	TWSC intersection on a six-lane street with U-turns and pedestrians	Operational

TWSC EXAMPLE PROBLEM 3: FLARED APPROACHES AND MEDIAN STORAGE

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Major street with two lanes in each direction, minor street with one lane on each approach that flares with storage for one vehicle in the flare area, and median storage for two vehicles at one time available for minor-street through and left-turn movements;
- Level grade on all approaches;
- Percent heavy vehicles on all approaches = 10%;
- Peak hour factor on all approaches = 0.92;
- Length of analysis period = 0.25 h; and
- Volumes and lane configurations as shown in Exhibit 32-8.

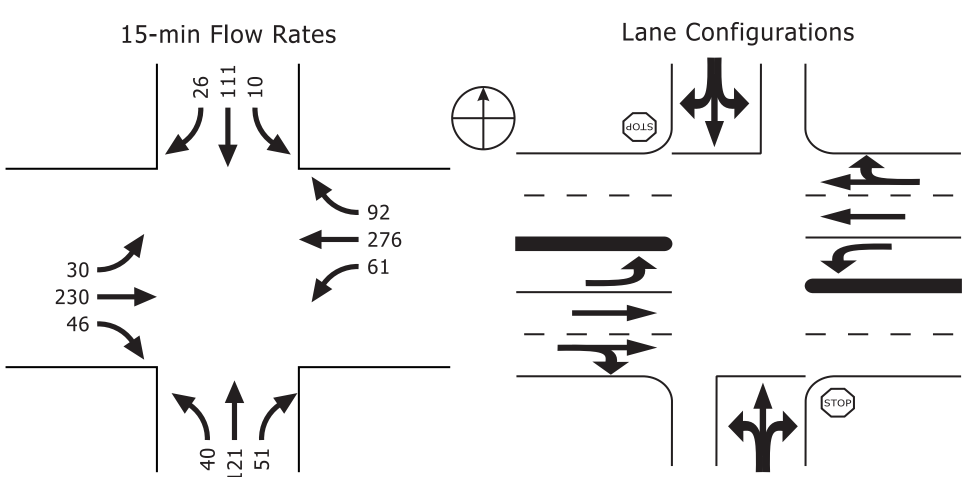


Exhibit 32-8 TWSC Example Problem 3: Volumes and Lane Configurations

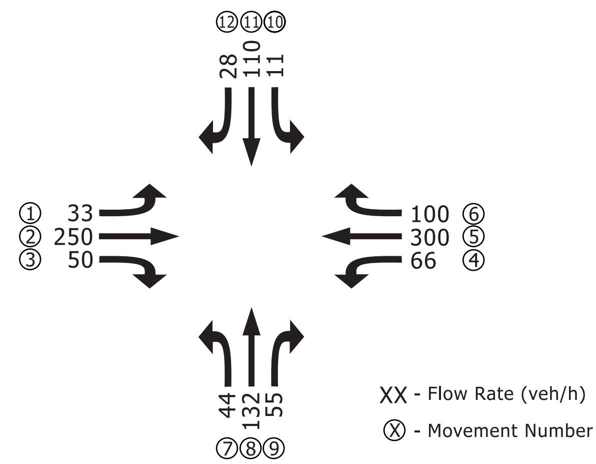
Comments

All relevant input parameters are known, so no default values are needed or used.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Because hourly volumes and a peak hour factor have been provided, each hourly volume is divided by the peak hour factor to determine a peak 15-min flow rate (in veh/h) for each movement. They are shown in Exhibit 32-9.

Exhibit 32-9
TWSC Example Problem 3: Calculation of Peak 15-min Flow Rates, Movement Priorities



Step 3: Compute Conflicting Flow Rates

The conflicting flow rates for each minor movement at the intersection are computed according to the equations in Chapter 19. The conflicting flow for the eastbound major-street left-turn movement $v_{c,1}$ is computed according to Equation 19-2 as follows:

$$v_{c,1} = v_5 + v_6 + v_{16} = 300 + 100 + 0 = 400$$

Similarly, the conflicting flow for the westbound major-street left-turn movement $v_{c,4}$ is computed according to Equation 19-3 as follows:

$$v_{c,4} = v_2 + v_3 + v_{15} = 250 + 50 + 0 = 300$$

The conflicting flows for the northbound minor-street right-turn movement $v_{c,9}$ and southbound minor-street right-turn movement $v_{c,12}$ are computed using Equation 19-6 and Equation 19-7, respectively, as follows (with no U-turns and pedestrians, which allows the last three terms to be assigned zero):

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15}$$

$$= 0.5(250) + 0.5(50) + 0 + 0 + 0 = 150$$

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{1U} + v_{13} + v_{16}$$

$$= 0.5(300) + 0.5(100) + 0 + 0 + 0 = 200$$

Next, the conflicting flow for the northbound minor-street through movement $v_{c,8}$ is computed. Because two-stage gap acceptance is available for this movement, the conflicting flow rates shown in Stage I and Stage II must be computed separately. The conflicting flow for Stage I $v_{c,I,8}$ is computed as follows:

$$v_{c,I,8} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

$$= 2(33 + 0) + 250 + 0.5(50) + 0 = 341$$

The conflicting flow for Stage II $v_{c,II,8}$ is computed as follows:

$$v_{c,II,8} = 2(v_4 + v_{4U}) + v_5 + v_6 + v_{16}$$

$$= 2(66 + 0) + 300 + 100 + 0 = 532$$

The total conflicting flow for the northbound through movement $v_{c,8}$ is computed as follows:

$$v_{c,8} = v_{c,I,8} + v_{c,II,8} = 341 + 532 = 873$$

Similarly, the conflicting flow for the southbound minor-street through movement $v_{c,11}$ is computed in two stages as follows:

$$v_{c,I,11} = 2(66 + 0) + 300 + 0.5(100) + 0 = 482$$

$$v_{c,II,11} = 2(33 + 0) + 250 + 50 + 0 = 366$$

$$v_{c,11} = 482 + 366 = 848$$

Next, the conflicting flow for the northbound minor-street left-turn movement $v_{c,7}$ is computed. Because two-stage gap acceptance is available for this movement, the conflicting flow rates shown in Stage I and Stage II must be computed separately. The conflicting flow for Stage I $v_{c,I,7}$ is computed with Equation 19-20 as follows:

$$v_{c,I,7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

$$v_{c,I,7} = 2(33 + 0) + 250 + 0.5(50) + 0 = 341$$

The conflicting flow for Stage II $v_{c,II,7}$ is computed with Equation 19-26 as follows:

$$v_{c,II,7} = 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$

$$= 2(66 + 0) + 0.5(300) + 0.5(110) + 0 = 337$$

The total conflicting flow for the northbound left-turn movement $v_{c,7}$ is computed as follows:

$$v_{c,7} = v_{c,I,7} + v_{c,II,7}$$

$$v_{c,7} = 341 + 337 = 678$$

Similarly, the conflicting flow for the southbound minor-street left-turn movement $v_{c,10}$ is computed in two stages as follows:

$$v_{c,I,10} = 2(66 + 0) + 300 + 0.5(100) + 0 = 482$$

$$v_{c,II,10} = 2(33 + 0) + 0.5(250) + 0.5(132) + 0 = 257$$

$$v_{c,10} = 482 + 257 = 739$$

Step 4: Determine Critical Headways and Follow-Up Headways

The critical headway for each minor movement is computed beginning with the base critical headway given in Exhibit 19-10. The base critical headway for each movement is then adjusted according to Equation 19-30. The critical headways for the eastbound and westbound major-street left turns $t_{c,1}$ and $t_{c,4}$ (in this case $t_{c,1} = t_{c,4}$), are computed as follows:

$$t_{c,1} = t_{c,4} = t_{c,base} + t_{c,HV} P_{HV} + t_{c,G} G - t_{3,LT}$$

$$t_{c,1} = t_{c,4} = 4.1 + 2.0(0.1) + 0(0) - 0 = 4.3$$

Next, the critical headways for the northbound and southbound minor-street right-turn movements $t_{c,9}$ and $t_{c,12}$ (in this case $t_{c,9} = t_{c,12}$) are computed as follows:

$$t_{c,9} = t_{c,12} = 6.9 + 2.0(0.1) + 0.1(0) - 0 = 7.1$$

Next, the critical headways for the northbound and southbound minor-street through movements $t_{c,8}$ and $t_{c,11}$ (in this case, $t_{c,8} = t_{c,11}$) are computed. Because two-stage gap acceptance is available for these movements, the critical headways for Stage I and Stage II must be computed, along with the critical headways for these movements assuming single-stage gap acceptance. The critical headways for Stage I and Stage II $t_{c,I,8}$, $t_{c,II,8}$ and $t_{c,I,11}$, $t_{c,II,11}$ (in this case, $t_{c,I,8} = t_{c,II,8} = t_{c,I,11} = t_{c,II,11}$) are computed as follows:

$$t_{c,I,8} = t_{c,II,8} = t_{c,I,11} = t_{c,II,11} = 5.5 + 2.0(0.1) + 0.2(0) - 0 = 5.7$$

The critical headways for $t_{c,8}$ and $t_{c,11}$ (in this case, $t_{c,8} = t_{c,11}$), assuming single-stage gap acceptance, are computed as follows:

$$t_{c,8} = t_{c,11} = 6.5 + 2.0(0.1) + 0.2(0) - 0 = 6.7$$

Finally, the critical headways for the northbound and southbound minor-street left-turn movements $t_{c,7}$ and $t_{c,10}$ (in this case, $t_{c,7} = t_{c,10}$) are computed. Because two-stage gap acceptance is available for these movements, the critical headways for Stage I and Stage II must be computed, along with the critical headways for these movements assuming single-stage gap acceptance. The critical headways for Stage I and Stage II $t_{c,I,7}$, $t_{c,II,7}$ and $t_{c,I,10}$, $t_{c,II,10}$ (in this case, $t_{c,I,7} = t_{c,II,7} = t_{c,I,10} = t_{c,II,10}$) are computed as follows:

$$t_{c,I,7} = t_{c,II,7} = t_{c,I,10} = t_{c,II,10} = 6.5 + 2.0(0.1) + 0.2(0) - 0 = 6.7$$

The critical headways for $t_{c,7}$ and $t_{c,10}$ (in this case $t_{c,7} = t_{c,10}$), assuming single-stage gap acceptance, are computed as follows:

$$t_{c,7} = t_{c,10} = 7.5 + 2.0(0.1) + 0.2(0) - 0 = 7.7$$

The follow-up headway for each minor movement is computed beginning with the base follow-up headway given in Exhibit 19-11. The base follow-up headway for each movement is then adjusted according to Equation 19-31. The follow-up headways for the northbound and southbound major-street left-turn movements $t_{f,1}$ and $t_{f,4}$ (in this case, $t_{f,1} = t_{f,4}$) are computed as follows:

$$t_{f,1} = t_{f,4} = t_{f,base} + t_{f,HV} P_{HV}$$

$$t_{f,1} = t_{f,4} = 2.2 + 1.0(0.1) = 2.3$$

Next, the follow-up headways for the northbound and southbound minor-street right-turn movements $t_{f,9}$ and $t_{f,12}$ (in this case, $t_{f,9} = t_{f,12}$) are computed as follows:

$$t_{f,9} = t_{f,12} = 3.3 + 1.0(0.1) = 3.4$$

Next, the follow-up headways for the northbound and southbound minor-street through movements $t_{f,8}$ and $t_{f,11}$ (in this case, $t_{f,8} = t_{f,11}$) are computed as follows:

$$t_{f,8} = t_{f,11} = 4.0 + 1.0(0.1) = 4.1$$

Follow-up headways for the minor-street through and left-turn movements are computed for the movement as a whole. Follow-up headways are not broken up by stage, since they apply only to vehicles as they exit the approach and enter the intersection.

Finally, the follow-up headways for the northbound and southbound minor-street left-turn movements $t_{f,7}$ and $t_{f,10}$ (in this case, $t_{f,7} = t_{f,10}$), are computed as follows:

$$t_{f,7} = t_{f,10} = 3.5 + 1.0(0.1) = 3.6$$

Step 5: Compute Potential Capacities

Because no upstream signals are present, the procedure in Step 5a is followed.

The computation of a potential capacity for each movement provides the analyst with a definition of capacity under the assumed base conditions. The potential capacity will be adjusted in later steps to estimate the movement capacity for each movement. The potential capacity for each movement is a function of the conflicting flow rate, critical headway, and follow-up headway computed in the previous steps. The potential capacity for the northbound major-street left-turn $c_{p,1}$ is computed as follows:

$$c_{p,1} = v_{c,1} \frac{e^{-v_{c,1}t_{c,1}/3{,}600}}{1 - e^{-v_{c,1}t_{f,1}/3{,}600}} = 400 \frac{e^{-(400)(4.3)/3{,}600}}{1 - e^{-(400)(2.3)/3{,}600}} = 1{,}100$$

Similarly, the potential capacities for Movements 4, 9, and 12, $c_{p,4}$, $c_{p,9}$, and $c_{p,12}$, are computed as follows:

$$c_{p,4} = 300 \frac{e^{-(300)(4.3)/3{,}600}}{1 - e^{-(300)(2.3)/3{,}600}} = 1{,}202$$

$$c_{p,9} = 150 \frac{e^{-(150)(7.1)/3{,}600}}{1 - e^{-(150)(3.4)/3{,}600}} = 845$$

$$c_{p,12} = 200 \frac{e^{-(200)(7.1)/3{,}600}}{1 - e^{-(200)(3.4)/3{,}600}} = 783$$

Because the two-stage gap-acceptance adjustment procedure will be implemented for estimating the capacity of the minor-street movements, three potential capacity values must be computed for each of Movements 7, 8, 10, and 11. First, the potential capacity must be computed for Stage I, $c_{p,I,8}$, $c_{p,I,11}$, $c_{p,I,7}$, and $c_{p,I,10}$, for each movement as follows:

$$c_{p,I,8} = 341 \frac{e^{-(341)(5.7)/3{,}600}}{1 - e^{-(341)(4.1)/3{,}600}} = 618$$

$$c_{p,I,11} = 482 \frac{e^{-(482)(5.7)/3,600}}{1-e^{-(482)(4.1)/3,600}} = 532$$

$$c_{p,I,7} = 341 \frac{e^{-(341)(6.7)/3,600}}{1-e^{-(341)(3.6)/3,600}} = 626$$

$$c_{p,I,10} = 482 \frac{e^{-(482)(6.7)/3,600}}{1-e^{-(482)(3.6)/3,600}} = 514$$

Next, the potential capacity must be computed for Stage II for each movement $c_{p,II,8}$, $c_{p,II,11}$, $c_{p,II,7}$, and $c_{p,II,10}$ as follows:

$$c_{p,II,8} = 532 \frac{e^{-(532)(5.7)/3,600}}{1-e^{-(532)(4.1)/3,600}} = 504$$

$$c_{p,II,11} = 366 \frac{e^{-(366)(5.7)/3,600}}{1-e^{-(366)(4.1)/3,600}} = 601$$

$$c_{p,II,7} = 337 \frac{e^{-(337)(6.7)/3,600}}{1-e^{-(337)(3.6)/3,600}} = 629$$

$$c_{p,II,10} = 257 \frac{e^{-(257)(6.7)/3,600}}{1-e^{-(257)(3.6)/3,600}} = 703$$

Finally, the potential capacity must be computed assuming single-stage gap acceptance for each movement, $c_{p,8}$, $c_{p,11}$, $c_{p,7}$, and $c_{p,10}$, as follows:

$$c_{p,8} = 873 \frac{e^{-(873)(6.7)/3,600}}{1-e^{-(873)(4.1)/3,600}} = 273$$

$$c_{p,11} = 848 \frac{e^{-(848)(6.7)/3,600}}{1-e^{-(848)(4.1)/3,600}} = 283$$

$$c_{p,7} = 678 \frac{e^{-(678)(7.7)/3,600}}{1-e^{-(678)(3.6)/3,600}} = 323$$

$$c_{p,10} = 739 \frac{e^{-(739)(7.7)/3,600}}{1-e^{-(739)(3.6)/3,600}} = 291$$

Steps 6–9: Compute Movement Capacities

Because no pedestrians are present, the procedures given in Chapter 19 are followed.

Steps 6: Rank 1 Movement Capacity

There is no computation for this step.

Step 7: Rank 2 Movement Capacity

Step 7a: Movement Capacity for Major-Street Left-Turn Movement

The movement capacity of each Rank 2 major-street left-turn movement is equal to its potential capacity:

$$c_{m,1} = c_{p,1} = 1{,}100$$

$$c_{m,4} = c_{p,4} = 1{,}202$$

Step 7b: Movement Capacity for Minor-Street Right-Turn Movement

The movement capacity of each minor-street right-turn movement is equal to its potential capacity:

$$c_{m,9} = c_{p,9} = 845$$

$$c_{m,12} = c_{p,12} = 783$$

Step 7c: Movement Capacity for Major-Street U-Turn Movement

No U-turns are present, so this step is skipped.

Step 7d: Effect of Major-Street Shared Through and Left-Turn Lane

Separate major-street left-turn lanes are provided, so this step is skipped.

Step 8: Rank 3 Movement Capacity

The movement capacity of each Rank 3 movement is equal to its potential capacity, factored by any impedance due to conflicting pedestrian or vehicular movements.

Step 8a: Rank 3 Movement Capacity for One-Stage Movements

As there are no pedestrians assumed at this intersection, the Rank 3 movements will be impeded only by other vehicular movements. Specifically, the Rank 3 movements will be impeded by major-street left-turning traffic, and as a first step in determining the impact of this impedance, the probability that these movements will operate in a queue-free state must be computed according to Equation 19-42:

$$p_{0,1} = 1 - \frac{v_1}{c_{m,1}} = 1 - \frac{33}{1{,}100} = 0.970$$

$$p_{0,4} = 1 - \frac{v_4}{c_{m,4}} = 1 - \frac{66}{1{,}202} = 0.945$$

Next, by using the probabilities computed above, capacity adjustment factors f_8 and f_{11} can be computed according to Equation 19-46:

$$f_8 = f_{11} = p_{0,1} p_{0,4} = (0.970)(0.945) = 0.917$$

Finally, under the single-stage gap-acceptance assumption, the movement capacities $c_{m,8}$ and $c_{m,11}$ can be computed according to Equation 19-47:

$$c_{m,8} = (c_{p,8}) f_8 = (273)(0.917) = 250$$

$$c_{m,11} = (c_{p,11}) f_{11} = (283)(0.917) = 260$$

Movements 8 and 11 will operate under two-stage gap acceptance. Therefore, the capacity adjustment procedure for estimating the capacity of Stage I and Stage II of these movements must be completed.

To begin the process of estimating Stage I and Stage II movement capacities, the probability of queue-free states on conflicting Rank 2 movements calculated above are entered into Equation 19-46 as before, but this time capacity adjustment factors are estimated for each individual stage as follows:

$$f_{I,8} = p_{0,1} = 0.970$$

$$f_{I,11} = p_{0,4} = 0.945$$

$$f_{II,8} = p_{0,4} = 0.945$$

$$f_{II,11} = p_{0,1} = 0.970$$

The Stage I movement capacities are then computed as follows:

$$c_{m,I,8} = (c_{p,I,8}) f_{I,8} = (618)(0.970) = 599$$

$$c_{m,I,11} = (c_{p,I,11}) f_{I,11} = (532)(0.945) = 503$$

The Stage II movement capacities are then computed as follows:

$$c_{m,II,8} = (c_{p,II,8}) f_{II,8} = (504)(0.945) = 476$$

$$c_{m,II,11} = (c_{p,II,11}) f_{II,11} = (601)(0.970) = 583$$

Step 8b: Rank 3 Capacity for Two-Stage Movements

The two-stage gap-acceptance procedure will result in a total capacity estimate for Movements 8 and 11. To begin the procedure, an adjustment factor a must be computed for each movement by using Equation 19-48, under the assumption that there is storage for two vehicles in the median refuge area; thus, $n_m = 2$:

$$a_8 = a_{11} = 1 - 0.32e^{-1.3\sqrt{n_m}} = 1 - 0.32e^{-1.3\sqrt{2}} = 0.949$$

Next, an intermediate variable, y, must be computed for each movement by using Equation 19-49:

$$y_8 = \frac{c_{m,I,8} - c_{m,8}}{c_{m,II,8} - v_1 - c_{m,8}} = \frac{599 - 250}{476 - 33 - 250} = 1.808$$

$$y_{11} = \frac{c_{m,I,11} - c_{m,11}}{c_{m,II,11} - v_4 - c_{m,11}} = \frac{503 - 260}{583 - 66 - 260} = 0.946$$

Finally, the total capacity for each movement $c_{T,8}$ and $c_{T,11}$ is computed according to Equation 19-50, since $y \neq 1$:

$$c_{m,T,8} = \frac{a_8}{y_8^{n_m+1} - 1}\left[y_8\left(y_8^{n_m} - 1\right)\left(c_{m,II,8} - v_1\right) + \left(y_8 - 1\right)c_{m,8}\right]$$

$$= \frac{0.949}{1.808^{2+1} - 1}\left[(1.808)\left(1.808^2 - 1\right)(476 - 33) + (1.808 - 1)(250)\right] = 390$$

$$c_{m,T,11} = \frac{a_{11}}{y_{11}^{n_m+1}-1}\left[y_{11}\left(y_{11}^{n_m}-1\right)\left(c_{m,II,11}-v_4\right)+\left(y_{11}-1\right)c_{m,11}\right]$$

$$= \frac{0.949}{0.946^{2+1}-1}\left[(0.946)\left(0.946^2-1\right)(583-66)+(0.946-1)(260)\right]=405$$

Step 9: Rank 4 Movement Capacity

Step 9a: Rank 4 Capacity for One-Stage Movements

The vehicle impedance effects for Rank 4 movements will first be estimated by assuming single-stage gap acceptance. Rank 4 movements are impeded by all of the same movements impeding Rank 2 and Rank 3 movements with the addition of impedances due to the minor-street crossing movements and minor-street right-turn movements. The probability that these movements will operate in a queue-free state must be incorporated into the procedure.

The probability that the minor-street right-turn movement will operate in a queue-free state, $p_{0,9}$ and $p_{0,12}$, can be computed as follows:

$$p_{0,9} = 1-\frac{v_9}{c_{m,9}} = 1-\frac{55}{845} = 0.935$$

$$p_{0,12} = 1-\frac{v_{12}}{c_{m,12}} = 1-\frac{28}{783} = 0.964$$

To compute p', the probability that both the major-street left-turn movements and the minor-street crossing movements will operate in a queue-free state simultaneously, the analyst must first compute $p_{0,k}$, which is done in the same manner as the computation of $p_{0,j}$, except that k represents Rank 3 movements. Therefore, the values for $p_{0,k}$, are computed as follows:

$$p_{0,8} = 1-\frac{v_8}{c_{m,T,8}} = 1-\frac{132}{390} = 0.662$$

$$p_{0,11} = 1-\frac{v_{11}}{c_{m,T,11}} = 1-\frac{110}{405} = 0.728$$

Next, the analyst must compute p'', which, under the single-stage gap-acceptance assumption, is simply the product of f_j and $p_{0,k}$. The value for $f_8 = f_{11}$ = 0.917 is as computed above. The value for $p_{0,11}$ is computed by using the total capacity for Movement 11 calculated in the previous step:

$$p_7'' = (p_{0,11})(f_{11}) = (0.728)(0.917) = 0.668$$

$$p_{10}'' = (p_{0,8})(f_8) = (0.662)(0.917) = 0.607$$

With the values for p'', the probability of a simultaneous queue-free state for each movement can be computed by using Equation 19-52 as follows:

$$p_7' = 0.65p_7'' - \frac{p_7''}{p_7''+3} + 0.6\sqrt{p_7''} = 0.65(0.668) - \frac{0.668}{0.668+3} + 0.6\sqrt{0.668} = 0.742$$

$$p_{10}' = 0.65(0.607) - \frac{0.607}{0.607 + 3} + 0.6\sqrt{0.607} = 0.694$$

Next, with the probabilities computed above, capacity adjustment factors f_7 and f_{10} can be computed according to Equation 19-53:

$$f_7 = (p_7')(p_{0,12}) = (0.742)(0.964) = 0.715$$

$$f_{10} = (p_{10}')(p_{0,9}) = (0.694)(0.935) = 0.649$$

Finally, under the single-stage gap-acceptance assumption, the movement capacities $c_{m,7}$ and $c_{m,10}$ can be computed according to Equation 19-54:

$$c_{m,7} = (c_{p,7})f_7 = (323)(0.715) = 231$$

$$c_{m,10} = (c_{p,10})f_{10} = (291)(0.649) = 189$$

Step 9b: Rank 4 Capacity for Two-Stage Movements

Similar to the minor-street crossing movements at this intersection, Movements 7 and 10 will also operate under two-stage gap acceptance. Therefore, the capacity adjustment procedure for estimating the capacity of Stage I and Stage II of these movements must be completed.

Under the assumption of two-stage gap acceptance with a median refuge area, the minor-street left-turn movements operate as Rank 3 movements in each individual stage of completing the left-turn maneuver. Therefore, to begin the process of estimating two-stage movement capacities, the probabilities of queue-free states on conflicting Rank 2 movements for Stage I of the minor-street left-turn movement are entered into Equation 19-46, and capacity adjustment factors for Stage I are computed as follows:

$$f_{I,7} = p_{0,1} = 0.970$$

$$f_{I,10} = p_{0,4} = 0.945$$

The Stage I movement capacities can then be computed as follows:

$$c_{m,I,7} = (c_{p,I,7})f_{I,7} = (626)(0.970) = 607$$

$$c_{m,I,10} = (c_{p,I,10})f_{I,10} = (514)(0.945) = 486$$

Next, the probabilities of queue-free states on conflicting Rank 2 movements for Stage II of the minor-street left-turn movement are entered into Equation 19-46. However, before estimating these probabilities, the probability of a queue-free state for the first stage of the minor-street crossing movement must be estimated as it impedes Stage II of the minor-street left-turn movement. These probabilities are estimated with Equation 19-42:

$$p_{0,I,8} = 1 - \frac{v_8}{c_{m,I,8}} = 1 - \frac{132}{599} = 0.780$$

$$p_{0,I,11} = 1 - \frac{v_{11}}{c_{m,I,11}} = 1 - \frac{110}{503} = 0.781$$

The capacity adjustment factors for Stage II are then computed as follows:

$$f_{II,7} = (p_{0,4})(p_{0,12})(p_{0,I,11}) = (0.945)(0.964)(0.781) = 0.711$$

$$f_{II,10} = (p_{0,1})(p_{0,9})(p_{0,I,8}) = (0.970)(0.935)(0.780) = 0.707$$

Finally, the movement capacities for Stage II are computed as follows:

$$c_{m,II,7} = (c_{p,II,7})f_{II,7} = (629)(0.711) = 447$$

$$c_{m,II,10} = (703)(0.707) = 497$$

The final result of the two-stage gap-acceptance procedure will be a total capacity estimate for Movements 7 and 10. To begin the procedure, an adjustment factor, a, must be computed for each movement by using Equation 19-55, under the assumption that there is storage for two vehicles in the median refuge area; thus, $n_m = 2$:

$$a_7 = a_{10} = 1 - 0.32e^{-1.3\sqrt{n_m}} = 1 - 0.32e^{-1.3\sqrt{2}} = 0.949$$

Next, an intermediate variable, y, must be computed for each movement by using Equation 19-56:

$$y_7 = \frac{c_{m,I,7} - c_{m,7}}{c_{m,II,7} - v_1 - c_{m,7}} = \frac{607 - 231}{447 - 33 - 231} = 2.055$$

$$y_{10} = \frac{c_{m,I,10} - c_{m,10}}{c_{m,II,10} - v_4 - c_{m,10}} = \frac{486 - 189}{497 - 66 - 189} = 1.227$$

Finally, the total capacity for each movement, $c_{T,8}$ and $c_{T,11}$, is computed according to Equation 19-57, as $y \neq 1$:

$$c_{T,7} = \frac{a_7}{y_7^{\,n_m+1} - 1}\left[y_7\left(y_7^{\,n_m} - 1\right)\left(c_{m,II,7} - v_1\right) + \left(y_7 - 1\right)c_{m,7}\right]$$

$$= \frac{0.949}{2.055^{2+1} - 1}\left[(2.055)\left(2.055^2 - 1\right)(447 - 33) + (2.055 - 1)(231)\right] = 369$$

$$c_{T,10} = \frac{a_{10}}{y_{10}^{\,n_m+1} - 1}\left[y_{10}\left(y_{10}^{\,n_m} - 1\right)\left(c_{m,II,10} - v_4\right) + \left(y_{10} - 1\right)c_{m,10}\right]$$

$$= \frac{0.949}{1.227^{2+1} - 1}\left[(1.227)\left(1.227^2 - 1\right)(497 - 66) + (1.227 - 1)(189)\right] = 347$$

Step 10: Compute Final Capacity Adjustments

In this example problem, several final capacity adjustments must be made to account for the effect of the shared lanes and the flared lanes on the minor-street approaches. Initially, the shared-lane capacities for each of the minor-street approaches must be computed on the assumption of no flared lanes; then the effects of the flare can be incorporated to compute an actual capacity for each minor-street approach.

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

In this example, both minor-street approaches have single-lane entries, meaning that all movements on the minor street share one lane. The shared-lane capacities for the minor-street approaches are computed according to Equation 19-59:

$$c_{SH,NB} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)} = \frac{v_7 + v_8 + v_9}{\frac{v_7}{c_{m,7}} + \frac{v_8}{c_{m,8}} + \frac{v_9}{c_{m,9}}} = \frac{44 + 132 + 55}{\frac{44}{369} + \frac{132}{390} + \frac{55}{845}} = 442$$

$$c_{SH,SB} = \frac{11 + 110 + 28}{\frac{11}{347} + \frac{110}{405} + \frac{28}{783}} = 439$$

Step 10b: Compute Flared Minor-Street Lane Effects

In this example, the capacity of each minor-street approach will be greater than the shared capacities computed in the previous step due to the shared-lane condition on each approach. On each approach, it is assumed that one vehicle at a time can queue in the flared area; therefore, $n = 1$.

First, the analyst must estimate the average queue length for each movement sharing the lane on each approach. Required input data for this estimation include the flow rates and control delays for each movement. While the flow rates are known input data, the control delays have not yet been computed. Therefore, the control delays for each movement, assuming a 15-min analysis period and separate lanes for each movement, must first be computed according to Equation 19-64:

$$d_7 = \frac{3{,}600}{c_{m,7}} + 900T\left[\frac{v_7}{c_{m,7}} - 1 + \sqrt{\left(\frac{v_7}{c_{m,7}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,7}}\right)\left(\frac{v_7}{c_{m,7}}\right)}{450T}}\right] + 5$$

$$d_7 = \frac{3{,}600}{369} + 900(0.25)\left[\frac{44}{369} - 1 + \sqrt{\left(\frac{44}{369} - 1\right)^2 + \frac{\left(\frac{3{,}600}{369}\right)\left(\frac{44}{369}\right)}{450(0.25)}}\right] + 5 = 16.07$$

$$d_8 = \frac{3{,}600}{390} + 900(0.25)\left[\frac{132}{390} - 1 + \sqrt{\left(\frac{132}{390} - 1\right)^2 + \frac{\left(\frac{3{,}600}{390}\right)\left(\frac{132}{390}\right)}{450(0.25)}}\right] + 5 = 18.88$$

$$d_9 = \frac{3{,}600}{845} + 900(0.25)\left[\frac{55}{845} - 1 + \sqrt{\left(\frac{55}{845} - 1\right)^2 + \frac{\left(\frac{3{,}600}{845}\right)\left(\frac{55}{845}\right)}{450(0.25)}}\right] + 5 = 9.57$$

$$d_{10} = \frac{3{,}600}{347} + 900(0.25)\left[\frac{11}{347} - 1 + \sqrt{\left(\frac{11}{347} - 1\right)^2 + \frac{\left(\frac{3{,}600}{347}\right)\left(\frac{11}{347}\right)}{450(0.25)}}\right] + 5 = 15.71$$

$$d_{11} = \frac{3{,}600}{405} + 900(0.25)\left[\frac{110}{405} - 1 + \sqrt{\left(\frac{110}{405} - 1\right)^2 + \frac{\left(\frac{3{,}600}{405}\right)\left(\frac{110}{405}\right)}{450(0.25)}}\right] + 5 = 17.17$$

$$d_{12} = \frac{3{,}600}{783} + 900(0.25)\left[\frac{28}{783} - 1 + \sqrt{\left(\frac{28}{783} - 1\right)^2 + \frac{\left(\frac{3{,}600}{783}\right)\left(\frac{28}{783}\right)}{450(0.25)}}\right] + 5 = 9.77$$

In this example, all movements on the minor-street approach share one lane; therefore, the average queue lengths for each minor-street movement are computed as follows:

$$Q_{sep,7} = \frac{d_{sep,7}v_{sep,7}}{3{,}600} = \frac{(16.07)(44)}{3{,}600} = 0.20$$

$$Q_{sep,8} = \frac{(18.88)(132)}{3{,}600} = 0.69$$

$$Q_{sep,9} = \frac{(9.57)(55)}{3{,}600} = 0.15$$

$$Q_{sep,10} = \frac{(15.71)(11)}{3{,}600} = 0.05$$

$$Q_{sep,11} = \frac{(17.17)(110)}{3{,}600} = 0.53$$

$$Q_{sep,12} = \frac{(9.77)(28)}{3{,}600} = 0.08$$

Next, the required length of the storage area so that each approach would operate effectively as separate lanes is computed with Equation 19-61:

$$n_{Max,NB} = \underset{NB}{\text{Max}}\left[\text{round}\left(Q_{sep,7} + 1\right), \text{round}\left(Q_{sep,8} + 1\right), \text{round}\left(Q_{sep,9} + 1\right)\right]$$

$$n_{Max,NB} = \underset{NB}{\text{Max}}\left[\text{round}(0.20 + 1), \text{round}(0.69 + 1), \text{round}(0.15 + 1)\right] = 2$$

$$n_{Max,SB} = \underset{SB}{\text{Max}}\left[\text{round}(0.05 + 1), \text{round}(0.53 + 1), \text{round}(0.08 + 1)\right] = 2$$

The next step involves estimating separate lane capacities, with consideration of the limitation of the amount of right-turn traffic that could actually move into a separate right-turn lane given a queue before the location of the flare. To compute separate lane capacities, the shared-lane capacities of the through plus left-turn movement on each approach must first be estimated according to Equation 19-59:

$$c_{L+TH,NB} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)} = \frac{v_7 + v_8}{\frac{v_7}{c_{m,7}} + \frac{v_8}{c_{m,8}}} = \frac{44+132}{\frac{44}{369}+\frac{132}{390}} = 385$$

$$c_{L+TH,SB} = \frac{v_{10} + v_{11}}{\frac{v_{10}}{c_{m,10}} + \frac{v_{11}}{c_{m,11}}} = \frac{11+110}{\frac{11}{347}+\frac{110}{405}} = 399$$

Then, the capacity of the separate lane condition c_{sep} for each approach can be computed according to Equation 19-62:

$$c_{sep,NB} = \text{Min}\left[c_{m,9}\left(1+\frac{v_{L+TH,NB}}{v_9}\right), c_{L+TH,NB}\left(1+\frac{v_9}{v_{L+TH,NB}}\right)\right]$$

$$= \text{Min}\left[845\left(1+\frac{44+132}{55}\right), 385\left(1+\frac{55}{44+132}\right)\right] = 505$$

$$c_{sep,SB} = \text{Min}\left[c_{m,12}\left(1+\frac{v_{L+TH,SB}}{v_{12}}\right), c_{L+TH,SB}\left(1+\frac{v_{12}}{v_{L+TH,SB}}\right)\right]$$

$$= \text{Min}\left[783\left(1+\frac{11+110}{28}\right), 399\left(1+\frac{28}{11+110}\right)\right] = 491$$

Finally, the capacities of the flared minor-street lanes are computed according to Equation 19-63:

$$c_{R,NB} = \begin{cases} \left(c_{sep,NB} - c_{SH,NB}\right)\frac{n_R}{n_{Max}} + c_{SH,NB} & \text{if } n_R \le n_{Max} \\ c_{sep,NB} & \text{if } n_R > n_{Max} \end{cases}$$

Because $n_R = 1$ and $n_{Max} = 2$, the first condition is evaluated:

$$c_{R,NB} = (505 - 442)\frac{1}{2} + 442 = 474$$

Similarly,

$$c_{R,SB} = (491 - 439)\frac{1}{2} + 439 = 465$$

Step 11: Compute Control Delay

The control delay computation for any movement includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The control delays for the major-street left-turn movements (Rank 2), d_1 and d_4, and the minor-street approaches, d_{NB} and d_{SB}, are computed as follows:

$$d_1 = \frac{3{,}600}{c_{m,1}} + 900T\left[\frac{v_1}{c_{m,1}} - 1 + \sqrt{\left(\frac{v_1}{c_{m,1}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,1}}\right)\left(\frac{v_1}{c_{m,1}}\right)}{450T}}\right] + 5$$

$$d_1 = \frac{3{,}600}{1{,}100} + 900(0.25)\left[\frac{33}{1{,}100} - 1 + \sqrt{\left(\frac{33}{1{,}100} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}100}\right)\left(\frac{33}{1{,}100}\right)}{450(0.25)}}\right] + 5 = 8.4$$

$$d_4 = \frac{3{,}600}{1{,}202} + 900(0.25)\left[\frac{66}{1{,}202} - 1 + \sqrt{\left(\frac{66}{1{,}202} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}202}\right)\left(\frac{66}{1{,}202}\right)}{450(0.25)}}\right] + 5 = 8.2$$

$$d_{NB} = \frac{3{,}600}{474} + 900(0.25)\left[\frac{231}{474} - 1 + \sqrt{\left(\frac{231}{474} - 1\right)^2 + \frac{\left(\frac{3{,}600}{474}\right)\left(\frac{231}{474}\right)}{450(0.25)}}\right] + 5 = 19.6$$

$$d_{SB} = \frac{3{,}600}{465} + 900(0.25)\left[\frac{149}{465} - 1 + \sqrt{\left(\frac{149}{465} - 1\right)^2 + \frac{\left(\frac{3{,}600}{465}\right)\left(\frac{149}{465}\right)}{450(0.25)}}\right] + 5 = 16.3$$

According to Exhibit 19-1, the levels of service (LOS) for the major-street left-turn movements and the minor-street approaches are as follows:

- Eastbound major-street left-turn (Movement 1): LOS A
- Westbound major-street left-turn (Movement 4): LOS A
- Northbound minor-street approach: LOS C
- Southbound minor-street approach: LOS C

Step 11b: Compute Control Delay to Rank 1 Movements

This step is not applicable as the major-street through movements v_2 and v_5 and westbound major-street left-turn movements v_1 and v_4 have exclusive lanes at this intersection.

Step 12: Compute Approach and Intersection Control Delay

The control delay for the eastbound approach $d_{A,EB}$ is computed as follows:

$$d_{A,EB} = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l} = \frac{(0)(50) + (0)(250) + (8.2)(33)}{50 + 250 + 33} = 0.8$$

The control delay for the westbound approach $d_{A,WB}$ is computed according to the same equation as for the eastbound approach:

$$d_{A,WB} = \frac{(0)(100) + (0)(300) + (8.4)(66)}{100 + 300 + 66} = 1.2$$

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB} + d_{A,SB} v_{A,SB}}{v_{A,EB} + v_{A,WB} + v_{A,NB} + v_{A,SB}}$$

$$d_I = \frac{(0.8)(333) + (1.2)(466) + (19.6)(231) + (16.3)(149)}{333 + 466 + 231 + 149} = 6.6$$

LOS is not defined for the intersection as a whole or for the major-street approaches.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for the major-street eastbound left-turn movement $Q_{95,1}$ is computed as follows:

$$Q_{95,1} \approx 900T \left[\frac{v_1}{c_{m,1}} - 1 + \sqrt{\left(\frac{v_1}{c_{m,1}} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{c_{m,1}} \right)\left(\frac{v_1}{c_{m,1}} \right)}{150T}} \right] \left(\frac{c_{m,1}}{3{,}600} \right)$$

$$Q_{95,1} \approx 900(0.25) \left[\frac{33}{1{,}100} - 1 + \sqrt{\left(\frac{33}{1{,}100} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{1{,}100} \right)\left(\frac{33}{1{,}100} \right)}{150(0.25)}} \right] \left(\frac{1{,}100}{3{,}600} \right) = 0.1$$

The result of 0.1 veh for the 95th percentile queue indicates that a queue of more than one vehicle will occur very infrequently for the eastbound major-street left-turn movement.

The 95th percentile queue length for the major-street westbound left-turn movement $Q_{95,4}$ is computed as follows:

$$Q_{95,4} \approx 900(0.25) \left[\frac{66}{1{,}202} - 1 + \sqrt{\left(\frac{66}{1{,}202} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{1{,}202} \right)\left(\frac{66}{1{,}202} \right)}{150(0.25)}} \right] \left(\frac{1{,}202}{3{,}600} \right) = 0.2$$

The result of 0.2 veh for the 95th percentile queue indicates that a queue of more than one vehicle will occur very infrequently for the westbound major-street left-turn movement.

The 95th percentile queue length for the northbound approach is computed by using the same formula, but, similar to the control delay computation, the shared-lane volume and shared-lane capacity must be used as shown:

$$Q_{95,NB} \approx 900(0.25)\left[\frac{231}{474}-1+\sqrt{\left(\frac{231}{474}-1\right)^2+\frac{\left(\frac{3{,}600}{474}\right)\left(\frac{231}{474}\right)}{150(0.25)}}\right]\left(\frac{474}{3{,}600}\right)=2.6$$

The result of 2.6 veh for the 95th percentile queue indicates that a queue of more than two vehicles will occur occasionally for the northbound approach.

The 95th percentile queue length for the southbound approach is computed by using the same formula, but, similar to the control delay computation, the shared-lane volume and shared-lane capacity must be used as shown:

$$Q_{95,SB} \approx 900(0.25)\left[\frac{149}{465}-1+\sqrt{\left(\frac{149}{465}-1\right)^2+\frac{\left(\frac{3{,}600}{465}\right)\left(\frac{149}{465}\right)}{150(0.25)}}\right]\left(\frac{465}{3{,}600}\right)=1.4$$

The result of 1.4 veh for the 95th percentile queue indicates that a queue of more than one vehicle will occur occasionally for the southbound approach.

Discussion

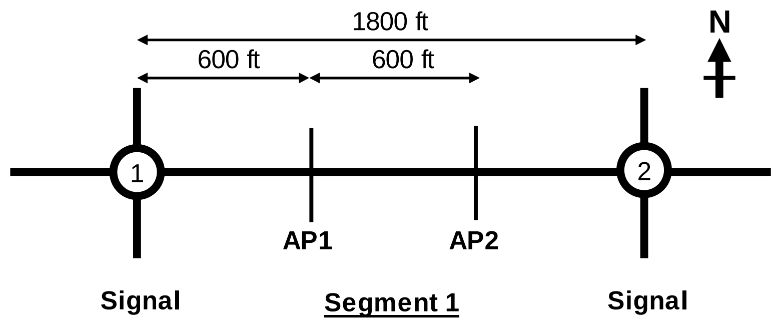
Overall, the results indicate that the four-leg TWSC intersection with two-stage gap-acceptance and flared minor-street approaches will operate satisfactorily with low delays for major-street movements and average delays for the minor-street approaches.

TWSC EXAMPLE PROBLEM 4: TWSC INTERSECTION WITHIN SIGNALIZED URBAN STREET SEGMENT

The Facts

This problem focuses on analyzing the performance of the TWSC intersection at Access Point 1 (AP1) from Example Problem 1 in Chapter 17, which looks at the automobile performance of the urban street segment bounded by two signalized intersections, as shown in Exhibit 32-10. The street has a four-lane cross section with two lanes in each direction.

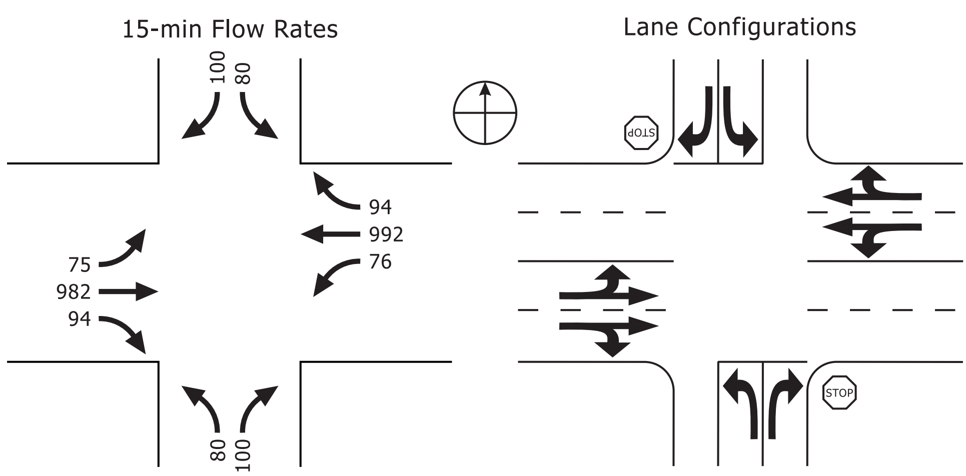
Exhibit 32-10
TWSC Example Problem 4: TWSC Intersection Within Signalized Urban Street Segment



From Example Problem 1 in Chapter 17, the following data are relevant:

- Major street with two lanes in each direction,
- Minor street with separate left-turn and right-turn lanes in each direction (through movements considered negligible) and STOP-controlled on minor-street approach,
- Level grade on all approaches,
- Percent heavy vehicles on all approaches = 1%,
- Length of analysis period = 0.25 h, and
- Flow rates and lane configurations as shown in Exhibit 32-11.

Exhibit 32-11
TWSC Example Problem 4: Flow Rates and Lane Configurations



The proportion time blocked and delay to through vehicles from the Chapter 17 methodology is as shown in Exhibit 32-12.

Exhibit 32-12
TWSC Example Problem 4: Movement-Based Access Point Output (from Chapter 17, Example Problem 1)

Access Point Data Segment 1 Movement:	EB L 1	EB T 2	EB R 3	WB L 4	WB T 5	WB R 6	NB L 7	NB T 8	NB R 9	SB L 10	SB T 11	SB R 12
Access Point Intersection No. 1												
1: Volume, veh/h	74.80	981.71	93.50	75.56	991.70	94.45	80.00	0.00	100.00	80.00	0.00	100.00
1: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1: Proportion time blocked	0.170			0.170			0.260	0.260	0.170	0.260	0.260	0.170
1: Delay to through vehicles, s/veh		0.163			0.164							
1: Prob. inside lane blocked by left		0.101			0.101							
1: Dist. from West/South signal, ft	600											
Access Point Intersection No. 2												
2: Volume, veh/h	75.56	991.70	94.45	74.80	981.71	93.50	80.00	0.00	100.00	80.00	0.00	100.00
2: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
2: Proportion time blocked	0.170			0.170			0.260	0.260	0.170	0.260	0.260	0.170
2: Delay to through vehicles, s/veh		0.164			0.163							
2: Prob. inside lane blocked by left		0.101			0.101							
2: Dist. from West/South signal, ft	1200											

Comments

Default values are needed for the saturation flow rates of the major-street through and right-turn movements for the analysis of shared or short major-street left-turn lanes:

- Major-street through movement, s_{i1} = 1,800 veh/h; and
- Major-street right-turn movement, s_{i2} = 1,500 veh/h.

All other input parameters are known.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Flow rates for each turning movement have been provided from the Chapter 17 methodology. They are assigned movement numbers as shown in Exhibit 32-13.

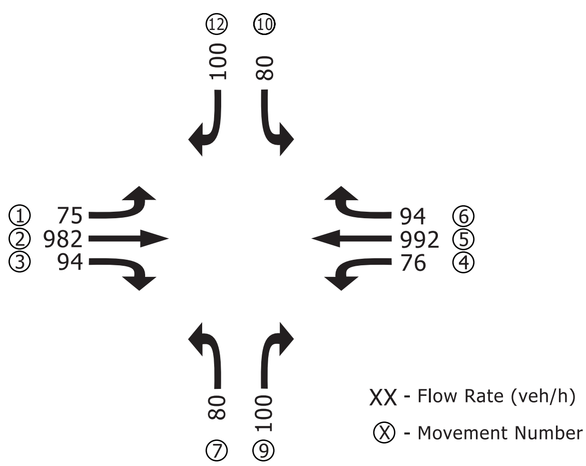


Exhibit 32-13
TWSC Example Problem 4: Calculation of Peak 15-min Flow Rates, Movement Priorities

Step 3: Compute Conflicting Flow Rates

Major-Street Left-Turn Movements (Rank 2—Movements 1 and 4)

The conflicting flows for the major-street left-turn movements are computed as follows:

$$v_{c,1} = v_5 + v_6 + v_{16} = 992 + 94 + 0 = 1{,}086$$

$$v_{c,4} = v_2 + v_3 + v_{15} = 982 + 94 + 0 = 1{,}076$$

Minor-Street Right-Turn Movements (Rank 2—Movements 9 and 12)

The conflicting flows for minor-street right-turn movements are computed as follows:

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15} = 0.5(982) + 0.5(94) + 0 + 0 + 0 = 538$$

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{1U} + v_{13} + v_{16} = 0.5(992) + 0.5(94) + 0 + 0 + 0 = 543$$

Major-Street U-Turn Movements (Rank 2—Movements 1U and 4U)

U-turns are assumed to be negligible.

Minor-Street Pedestrian Movements (Rank 2—Movements 13 and 14)

Minor-street pedestrian movements are assumed to be negligible.

Minor-Street Through Movements (Rank 3—Movements 8 and 11)

Because there are no minor-street through movements, this step can be skipped.

Minor-Street Left-Turn Movements (Rank 4—Movements 7 and 10)

Because the major street has four lanes without left-turn lanes or other possible median storage, the minor-street left-turn movement is assumed to be conducted in one stage. As a result, the conflicting flows for Stages I and II can be combined.

$$v_{c,7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15} + 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$

$$= 2(75+0) + 982 + 0.5(94) + 0 + 2(76+0) + 0.5(992) + 0.5(0) + 0 = 1{,}827$$

$$v_{c,10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16} + 2(v_1 + v_{1U}) + 0.5v_2 + 0.5v_8 + v_{14}$$

$$= 2(76+0) + 992 + 0.5(94) + 0 + 2(75+0) + 0.5(982) + 0.5(0) + 0 = 1{,}832$$

Step 4: Determine Critical Headways and Follow-Up Headways

Critical headways for each movement are computed as follows:

$$t_{c,x} = t_{c,base} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,1} = t_{c,4} = 4.1 + (2.0)(0.01) + 0 - 0 = 4.12$$

$$t_{c,9} = t_{c,12} = 6.9 + 2.0(0.01) + 0.1(0) - 0 = 6.92$$

$$t_{c,7} = t_{c,10} = 7.5 + 2.0(0.01) + 0.2(0) - 0 = 7.52$$

Follow-up headways for each movement are computed as follows:

$$t_{f,x} = t_{f,base} + t_{f,HV}P_{HV}$$

$$t_{f,1} = t_{f,4} = 2.2 + 1.0(0.01) = 2.21$$

$$t_{f,9} = t_{f,12} = 3.3 + 1.0(0.01) = 3.31$$

$$t_{f,7} = t_{f,10} = 3.5 + 1.0(0.01) = 3.51$$

Step 5: Compute Potential Capacities

Because upstream signals are present, Step 5b is used. The proportion time blocked for each movement x is given as $p_{b,x}$ and has been computed by the Chapter 17 procedure.

The flow for the unblocked period (no platoons) is determined by first computing the conflicting flow for each movement during the unblocked period (Equation 19-40). The minimum platooned flow rate $v_{c,min}$ over two lanes is assumed to be equal to $1{,}000N = 1{,}000(2) = 2{,}000$. The flow rate assumed to occur during the blocked period is calculated as follows:

$$1.5v_{c,\min}p_{b,1} = 1.5(2{,}000)(0.170) = 510$$

The value for $v_{c,1}$ = 1,086 exceeds this value, which indicates that some of the conflicting flow occurs in the unblocked period. Therefore, $v_{c,u,1}$ is calculated as follows:

$$v_{c,u,1} = \frac{v_{c,1} - 1.5v_{c,\min}p_{b,1}}{1 - p_{b,1}} = \frac{1{,}086 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 694$$

Similar calculations are made for the other movements as follows:

$$v_{c,u,4} = \frac{v_{c,4} - 1.5v_{c,\min}p_{b,4}}{1 - p_{b,4}} = \frac{1076 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 682$$

$$v_{c,u,9} = \frac{v_{c,9} - 1.5v_{c,\min}p_{b,9}}{1 - p_{b,9}} = \frac{538 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 34$$

$$v_{c,u,12} = \frac{v_{c,12} - 1.5v_{c,\min}p_{b,12}}{1 - p_{b,12}} = \frac{543 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 40$$

$$v_{c,u,7} = \frac{v_{c,7} - 1.5v_{c,\min}p_{b,7}}{1 - p_{b,7}} = \frac{1{,}827 - 1.5(2{,}000)(0.260)}{1 - 0.260} = 1{,}415$$

$$v_{c,u,10} = \frac{v_{c,10} - 1.5v_{c,\min}p_{b,10}}{1 - p_{b,10}} = \frac{1{,}832 - 1.5(2{,}000)(0.260)}{1 - 0.260} = 1{,}422$$

The potential capacity for each movement is then calculated with Equation 19-34 and Equation 19-35 (combined) as follows:

$$c_{p,1} = \left(1 - p_{b,1}\right)v_{c,u,1}\frac{e^{-v_{c,u,1}t_{c,1}/3{,}600}}{1 - e^{-v_{c,u,1}t_{f,1}/3{,}600}}$$

$$= \left(1 - 0.170\right)(694)\frac{e^{-(694)(4.12)/3{,}600}}{1 - e^{-(694)(2.21)/3{,}600}} = 750$$

$$c_{p,4} = \left(1 - 0.170\right)(682)\frac{e^{-(682)(4.12)/3{,}600}}{1 - e^{-(682)(2.21)/3{,}600}} = 758$$

$$c_{p,9} = \left(1 - 0.170\right)(34)\frac{e^{-(34)(6.92)/3{,}600}}{1 - e^{-(34)(3.31)/3{,}600}} = 859$$

$$c_{p,12} = \left(1 - 0.170\right)(40)\frac{e^{-(40)(6.92)/3{,}600}}{1 - e^{-(40)(3.31)/3{,}600}} = 851$$

$$c_{p,7} = \left(1 - 0.260\right)(1{,}415)\frac{e^{-(1415)(7.52)/3{,}600}}{1 - e^{-(1415)(3.51)/3{,}600}} = 73$$

$$c_{p,10} = \left(1 - 0.260\right)(1{,}422)\frac{e^{-(1422)(7.52)/3{,}600}}{1 - e^{-(1422)(3.51)/3{,}600}} = 72$$

Steps 6–9: Compute Movement Capacities

Because no pedestrians are present, the procedures given in Chapter 19 are followed.

Step 6: Rank 1 Movement Capacity

There is no computation for this step. The adjustment for the delay to through movements caused by left-turn movements in the shared left-through lane is accounted for by using adjustments provided later in this procedure.

Step 7: Rank 2 Movement Capacity

Step 7a: Movement Capacity for Major-Street Left-Turn Movement

The movement capacity of each Rank 2 major-street left-turn movement (1 and 4) is equal to its potential capacity as follows:

$$c_{m,1} = c_{p,1} = 750$$

$$c_{m,4} = c_{p,4} = 758$$

Step 7b: Movement Capacity for Minor-Street Right-Turn Movement

The movement capacity of each minor-street right-turn movement is equal to its potential capacity:

$$c_{m,9} = c_{p,9} = 859$$

$$c_{m,12} = c_{p,12} = 851$$

Step 7c: Movement Capacity for Major-Street U-Turn Movement

No U-turns are present, so this step is skipped.

Step 7d: Effect of Major-Street Shared Through and Left-Turn Lane

The probability that the major-street left-turning traffic will operate in a queue-free state, assuming that the left-turn movement occupies its own lane, is calculated with Equation 19-51 as follows:

$$p_{0,1} = 1 - \frac{v_1}{c_{m,1}} = 1 - \frac{75}{750} = 0.900$$

$$p_{0,4} = 1 - \frac{v_4}{c_{m,4}} = 1 - \frac{76}{758} = 0.900$$

However, for this problem the major-street left-turn movement shares a lane with the through movement. First, the combined degree of saturation for the major-street through and right-turn movements is calculated as follows (using default values for s):

$$x_{2+3} = \frac{v_2}{s_2} + \frac{v_3}{s_3} = \frac{982}{1{,}800} + \frac{94}{1{,}500} = 0.608$$

$$x_{5+6} = \frac{v_5}{s_5} + \frac{v_6}{s_6} = \frac{992}{1{,}800} + \frac{94}{1{,}500} = 0.614$$

Next, the probability that there will be no queue in the major-street shared lane $p^*_{0,j}$ is calculated according to the special case ($n_L = 0$) given as follows:

$$p^*_{0,1} = 1 - \frac{1 - p_{0,1}}{1 - x_{2+3}} = 1 - \frac{1 - 0.900}{1 - 0.608} = 0.745$$

$$p^*_{0,4} = 1 - \frac{1 - p_{0,4}}{1 - x_{5+6}} = 1 - \frac{1 - 0.900}{1 - 0.614} = 0.741$$

These values of $p^*_{0,1}$ and $p^*_{0,4}$ are used in lieu of $p_{0,1}$ and $p_{0,4}$ for the remaining calculations.

Step 8: Compute Movement Capacities for Rank 3 Movements

Step 8a: Rank 3 Movement Capacity for One-Stage Movements

Because there are no minor-street through movements, it is not necessary to compute the movement capacities for those movements. However, capacity adjustment factors f_8 and f_{11} are needed for subsequent steps and can be computed as follows:

$$f_8 = f_{11} = p^*_{0,1} p^*_{0,4} = (0.745)(0.741) = 0.552$$

Step 8b: Rank 3 Capacity for Two-Stage Movements

No two-stage movements are present, so this step is skipped.

Step 9: Compute Movement Capacities for Rank 4 Movements

Step 9a: Rank 4 Capacity for One-Stage Movements

The probabilities that the minor-street right-turn movements will operate in the queue-free state $p_{0,9}$ and $p_{0,12}$ are computed as follows:

$$p_{0,9} = 1 - \frac{v_9}{c_{m,9}} = 1 - \frac{100}{859} = 0.884$$

$$p_{0,12} = 1 - \frac{v_{12}}{c_{m,12}} = 1 - \frac{100}{851} = 0.882$$

To compute p', the probability that both the major-street left-turn movements and the minor-street crossing movements will operate in a queue-free state simultaneously, the analyst must first compute $p_{0,k}$, which is done in the same manner as the computation of $p_{0,j}$, except that k represents Rank 3 movements. Therefore, the values for $p_{0,k}$ are computed as follows:

$$p_{0,8} = 1 - \frac{v_8}{c_{m,8}} = 1 - 0 = 1$$

$$p_{0,11} = 1 - \frac{v_{11}}{c_{m,11}} = 1 - 0 = 1$$

Next, the analyst must compute p'', which, under the single-stage gap-acceptance assumption, is simply the product of f_j and $p_{0,k}$. The value for $f_8 = f_{11}$ = 0.552 is as computed above. The value for $p_{0,11}$ is computed by using the total capacity for Movement 11 calculated in the previous step:

$$p_7'' = (p_{0,11})(f_{11}) = (1)(0.552) = 0.552$$

$$p_{10}'' = (p_{0,8})(f_8) = (1)(0.552) = 0.552$$

By using the values for p'', the probability of a simultaneous queue-free state for each movement can be computed with Equation 19-52 as follows:

$$p_7' = 0.65 p_7'' - \frac{p_7''}{p_7''+3} + 0.6\sqrt{p_7''} = 0.65(0.552) - \frac{0.552}{0.552+3} + 0.6\sqrt{0.552} = 0.649$$

$$p_{10}' = 0.65(0.552) - \frac{0.552}{0.552+3} + 0.6\sqrt{0.552} = 0.649$$

Next, by using the probabilities computed above, capacity adjustment factors f_7 and f_{10} can be computed as follows:

$$f_7 = (p_7')(p_{0,12}) = (0.649)(0.882) = 0.572$$

$$f_{10} = (p_{10}')(p_{0,9}) = (0.649)(0.884) = 0.574$$

Finally, the movement capacities $c_{m,7}$ and $c_{m,10}$ can be computed as follows:

$$c_{m,7} = (c_{p,7}) f_7 = (73)(0.572) = 42$$

$$c_{m,10} = (c_{p,10}) f_{10} = (72)(0.574) = 41$$

Step 9b: Rank 4 Capacity for Two-Stage Movements

No two-stage movements are present, so this step is skipped.

Step 10: Final Capacity Adjustments

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

No shared lanes are present on the side street, so this step is skipped.

Step 10b: Compute Flared Minor-Street Lane Effects

No flared lanes are present, so this step is skipped.

Step 11: Compute Movement Control Delay

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The delay for each minor-street movement is calculated as follows:

$$d_1 = \frac{3{,}600}{750} + 900(0.25)\left[\frac{75}{750} - 1 + \sqrt{\left(\frac{75}{750} - 1\right)^2 + \frac{\left(\frac{3{,}600}{750}\right)\left(\frac{75}{750}\right)}{450(0.25)}}\right] + 5 = 10.3$$

$$d_4 = \frac{3{,}600}{758} + 900(0.25)\left[\frac{76}{758} - 1 + \sqrt{\left(\frac{76}{758} - 1\right)^2 + \frac{\left(\frac{3{,}600}{758}\right)\left(\frac{76}{758}\right)}{450(0.25)}}\right] + 5 = 10.3$$

$$d_9 = \frac{3{,}600}{859} + 900(0.25)\left[\frac{100}{859} - 1 + \sqrt{\left(\frac{100}{859} - 1\right)^2 + \frac{\left(\frac{3{,}600}{859}\right)\left(\frac{100}{859}\right)}{450(0.25)}}\right] + 5 = 9.7$$

$$d_{12} = \frac{3{,}600}{851} + 900(0.25)\left[\frac{100}{851} - 1 + \sqrt{\left(\frac{100}{851} - 1\right)^2 + \frac{\left(\frac{3{,}600}{851}\right)\left(\frac{100}{851}\right)}{450(0.25)}}\right] + 5 = 9.8$$

$$d_7 = \frac{3{,}600}{42} + 900(0.25)\left[\frac{80}{42} - 1 + \sqrt{\left(\frac{80}{42} - 1\right)^2 + \frac{\left(\frac{3{,}600}{42}\right)\left(\frac{80}{42}\right)}{450(0.25)}}\right] + 5 = 633$$

$$d_{10} = \frac{3{,}600}{41} + 900(0.25)\left[\frac{80}{41} - 1 + \sqrt{\left(\frac{80}{41} - 1\right)^2 + \frac{\left(\frac{3{,}600}{41}\right)\left(\frac{80}{41}\right)}{450(0.25)}}\right] + 5 = 657$$

According to Exhibit 19-1, the LOS for the major-street left-turn movements and the minor-street approaches are as follows:

- Eastbound major-street left turn (Movement 1): LOS B
- Westbound major-street left turn (Movement 4): LOS B
- Northbound minor-street right turn (Movement 9): LOS A
- Southbound minor-street right turn (Movement 12): LOS A
- Northbound minor-street left turn (Movement 7): LOS F
- Southbound minor-street left turn (Movement 10): LOS F

Step 11b: Compute Control Delay to Rank 1 Movements

The presence of a shared left through lane on the major street creates delay for Rank 1 movements (major-street through movements). Assuming that major-street through vehicles distribute equally across both lanes, then $v_{i,1} = v_2/N = 982/2$

= 491. The number of major-street turning vehicles in the shared lane is equal to the major-street left-turn flow rate; therefore, $v_{i,2}$ = 75.

The average delay to Rank 1 vehicles is therefore computed with Equation 19-65 as follows:

$$d_2 = \frac{\left(1 - p_{0,1}^*\right) d_1 \left(\frac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} = \frac{(1 - 0.745)(10.3)\left(\frac{491}{2}\right)}{491 + 75} = 1.1$$

Similarly, for the opposite direction, $v_{i,1} = v_5/N = 992/2 = 496$. The number of major-street turning vehicles in the shared lane is equal to the major-street left-turn flow rate; therefore, $v_{i,2}$ = 76.

$$d_5 = \frac{\left(1 - p_{0,4}^*\right) d_4 \left(\frac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} = \frac{(1 - 0.741)(10.3)\left(\frac{496}{2}\right)}{496 + 76} = 1.2$$

The procedures in Chapter 17 provide a better estimate of delay to major-street through vehicles: d_2 = 0.2 and d_5 = 0.2. These values account for the likelihood of major-street through vehicles shifting out of the shared left through lane to avoid being delayed by major-street left-turning vehicles. These values are used in the calculations in Step 12.

Step 12: Compute Approach and Intersection Control Delay

The control delay for each approach is computed as follows:

$$d_{A,EB} = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l} = \frac{(0)(94) + (1.1)(982) + (10.3)(75)}{94 + 982 + 75} = 1.6$$

$$d_{A,WB} = \frac{(0)(94) + (1.2)(992) + (10.3)(76)}{94 + 992 + 76} = 1.7$$

$$d_{A,NB} = \frac{(9.7)(100) + 0 + (633)(80)}{100 + 0 + 80} = 287$$

$$d_{A,SB} = \frac{(9.8)(100) + 0 + (657)(80)}{100 + 0 + 80} = 297$$

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB} + d_{A,SB} v_{A,SB}}{v_{A,EB} + v_{A,WB} + v_{A,NB} + v_{A,SB}}$$

$$d_I = \frac{(1.6)(1151) + (1.7)(1162) + (287)(180) + (297)(180)}{1151 + 1162 + 180 + 180} = 40.8$$

LOS is not defined for the intersection as a whole or for the major-street approaches. This fact is particularly important for this problem, as the assignment of LOS to the intersection as a whole would mask the severe LOS F condition on the minor-street left-turn movement.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for each movement is computed as follows:

$$Q_{95,1} \approx 900(0.25)\left[\frac{75}{750}-1+\sqrt{\left(\frac{75}{750}-1\right)^2+\frac{\left(\frac{3{,}600}{750}\right)\left(\frac{75}{750}\right)}{150(0.25)}}\right]\left(\frac{750}{3{,}600}\right)=0.3$$

$$Q_{95,4} \approx 900(0.25)\left[\frac{76}{758}-1+\sqrt{\left(\frac{76}{758}-1\right)^2+\frac{\left(\frac{3{,}600}{758}\right)\left(\frac{76}{758}\right)}{150(0.25)}}\right]\left(\frac{758}{3{,}600}\right)=0.3$$

$$Q_{95,9} \approx 900(0.25)\left[\frac{100}{859}-1+\sqrt{\left(\frac{100}{859}-1\right)^2+\frac{\left(\frac{3{,}600}{859}\right)\left(\frac{100}{859}\right)}{150(0.25)}}\right]\left(\frac{859}{3{,}600}\right)=0.4$$

$$Q_{95,12} \approx 900(0.25)\left[\frac{100}{851}-1+\sqrt{\left(\frac{100}{851}-1\right)^2+\frac{\left(\frac{3{,}600}{851}\right)\left(\frac{100}{851}\right)}{150(0.25)}}\right]\left(\frac{851}{3{,}600}\right)=0.4$$

$$Q_{95,7} \approx 900(0.25)\left[\frac{80}{42}-1+\sqrt{\left(\frac{80}{42}-1\right)^2+\frac{\left(\frac{3{,}600}{42}\right)\left(\frac{80}{42}\right)}{150(0.25)}}\right]\left(\frac{42}{3{,}600}\right)=8.3$$

$$Q_{95,10} \approx 900(0.25)\left[\frac{80}{41}-1+\sqrt{\left(\frac{80}{41}-1\right)^2+\frac{\left(\frac{3{,}600}{41}\right)\left(\frac{80}{41}\right)}{150(0.25)}}\right]\left(\frac{41}{3{,}600}\right)=8.4$$

The results indicate that queues of more than one vehicle will rarely occur for the major-street left-turn and minor-street right-turn movements. Longer queues are expected for the minor-street left-turn movements, and these queues are likely to be unstable under the significantly oversaturated conditions.

Discussion

The results indicate that Access Point 1 will operate over capacity (LOS F) for the minor-street left-turn movements. All other movements are expected to operate at LOS B or better, with low average delays and short queue lengths.

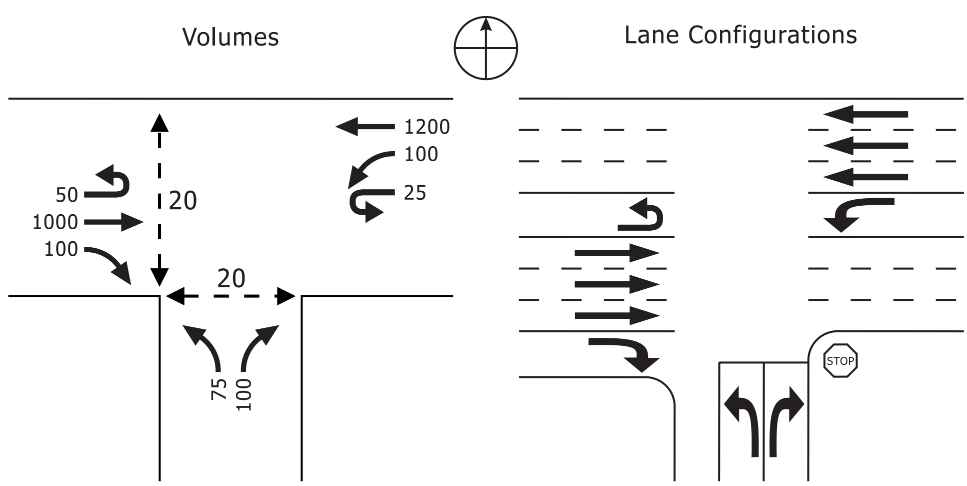
TWSC EXAMPLE PROBLEM 5: SIX-LANE STREET WITH U-TURNS AND PEDESTRIANS

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- T-intersection,
- Major street with three lanes in each direction,
- Minor street with separate left-turn and right-turn lanes and STOP control on the minor-street approach [minor-street left turns operate in two stages (room for storage of one vehicle)],
- Level grade on all approaches,
- Percent heavy vehicles on all approaches = 0%,
- Lane width = 12 ft,
- No other unique geometric considerations or upstream signal considerations,
- 20 p/h crossing both the west and south legs [each pedestrian is assumed to cross in its own group (i.e., independently)],
- Peak hour factor = 1.00,
- Length of analysis period = 0.25 h, and
- Hourly volumes and lane configurations as shown in Exhibit 32-14.

Exhibit 32-14
TWSC Example Problem 5: Volumes and Lane Configurations



Comments

The assumed walking speed of pedestrians is 3.5 ft/s.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Flow rates for each turning movement are the same as the peak hour volumes because the peak hour factor equals 1.0. These movements are assigned numbers as shown in Exhibit 32-15.

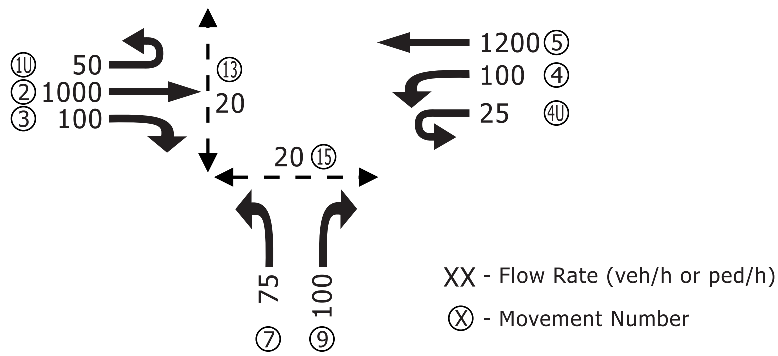


Exhibit 32-15
TWSC Example Problem 5: Calculation of Peak 15-min Flow Rates, Movement Priorities

Step 3: Compute Conflicting Flow Rates

Major-Street Left-Turn Movements (Rank 2—Movements 1 and 4)

The conflicting flow rate for the major-street left-turn movement is computed as follows:

$$v_{c,4} = v_2 + v_3 + v_{15} = 1{,}000 + 100 + 20 = 1{,}120$$

Minor-Street Right-Turn Movements (Rank 2—Movements 9 and 12)

The conflicting flow rate for the minor-street right-turn movement is computed as follows (dropping the v_3 term due to a separate major-street right-turn lane):

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15} = 0.5(1{,}000) + 0 + 0 + 20 = 520$$

Major-Street U-Turn Movements (Rank 2—Movements 1U and 4U)

The conflicting flow rates for the major-street U-turns are computed as follows (again dropping the v_3 term):

$$v_{c,1U} = 0.73v_5 + 0.73v_6 = 0.73(1{,}200) + 0 = 876$$

$$v_{c,4U} = 0.73v_2 + 0.73v_3 = 0.73(1{,}000) + 0 = 730$$

Minor-Street Left-Turn Movements (Rank 3—Movement 7)

The conflicting flow rate for Stage I of the minor-street left-turn movement is computed as follows (the v_3 term in these equations is assumed to be zero because of the right-turn lane on the major street):

$$v_{c,I,7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15} = 2(0 + 50) + 1{,}000 + 0 + 20 = 1{,}120$$

The conflicting flow rate for Stage II of the minor-street left-turn movement is computed as follows:

$$v_{c,II,7} = 2(v_4 + v_{4U}) + 0.4v_5 + 0.5v_{11} + v_{13}$$

$$v_{c,II,7} = 2(100 + 25) + 0.4(1{,}200) + 0 + 20 = 750$$

$$v_{c,7} = v_{c,I,7} + v_{c,II,7} = 1{,}120 + 750 = 1{,}870$$

Step 4: Determine Critical Headways and Follow-Up Headways

Critical headways for each minor movement are computed as follows:

$$t_{c,x} = t_{c,base} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,1U} = 5.6 + 0 + 0 - 0 = 5.6$$

$$t_{c,4} = 5.3 + 0 + 0 - 0 = 5.3$$

$$t_{c,4U} = 5.6 + 0 + 0 - 0 = 5.6$$

$$t_{c,9} = 7.1 + 0 + 0 - 0 = 7.1$$

$$t_{c,7} = 6.4 + 0 + 0 - 0.7 = 5.7$$

$$t_{c,I,7} = 7.3 + 0 + 0 - 0.7 = 6.6$$

$$t_{c,II,7} = 6.7 + 0 + 0 - 0.7 = 6.0$$

Follow-up headways for each minor movement are computed as follows:

$$t_{f,x} = t_{f,base} + t_{f,HV}P_{HV}$$

$$t_{f,1U} = 2.3 + 0 = 2.3$$

$$t_{f,4} = 3.1 + 0 = 3.1$$

$$t_{f,4U} = 2.3 + 0 = 2.3$$

$$t_{f,9} = 3.9 + 0 = 3.9$$

$$t_{f,7} = 3.8 + 0 = 3.8$$

Step 5: Compute Potential Capacities

Because no upstream signals are present, Step 5a is used. The potential capacity $c_{p,x}$ for each movement is computed as follows:

$$c_{p,1U} = v_{c,1U}\frac{e^{-v_{c,1U}t_{c,1U}/3{,}600}}{1-e^{-v_{c,1U}t_{f,1U}/3{,}600}} = 876\frac{e^{-(876)(5.6)/3{,}600}}{1-e^{-(876)(2.3)/3{,}600}} = 523$$

$$c_{p,4} = v_{c,4}\frac{e^{-v_{c,4}t_{c,4}/3{,}600}}{1-e^{-v_{c,4}t_{f,4}/3{,}600}} = 1{,}120\frac{e^{-(1120)(5.3)/3{,}600}}{1-e^{-(1120)(3.1)/3{,}600}} = 348$$

$$c_{p,4U} = v_{c,4U}\frac{e^{-v_{c,4U}t_{c,4U}/3{,}600}}{1-e^{-v_{c,4U}t_{f,4U}/3{,}600}} = 730\frac{e^{-(730)(5.6)/3{,}600}}{1-e^{-(730)(2.3)/3{,}600}} = 629$$

$$c_{p,9} = v_{c,9}\frac{e^{-v_{c,9}t_{c,9}/3{,}600}}{1-e^{-v_{c,9}t_{f,9}/3{,}600}} = 520\frac{e^{-(520)(7.1)/3{,}600}}{1-e^{-(520)(3.9)/3{,}600}} = 433$$

$$c_{p,7} = v_{c,7}\frac{e^{-v_{c,7}t_{c,7}/3{,}600}}{1-e^{-v_{c,7}t_{f,7}/3{,}600}} = 1{,}870\frac{e^{-(1870)(5.7)/3{,}600}}{1-e^{-(1870)(3.8)/3{,}600}} = 112$$

$$c_{p,I,7} = v_{c,I,7}\frac{e^{-v_{c,I,7}t_{c,7}/3{,}600}}{1-e^{-v_{c,I,7}t_{f,7}/3{,}600}} = 1{,}120\frac{e^{-(1120)(6.6)/3{,}600}}{1-e^{-(1120)(3.8)/3{,}600}} = 207$$

$$c_{p,II,7} = v_{c,II,7}\frac{e^{-v_{c,II,7}t_{c,7}/3,600}}{1-e^{-v_{c,II,7}t_{f,7}/3,600}} = 750\frac{e^{-(750)(6.0)/3,600}}{1-e^{-(750)(3.8)/3,600}} = 393$$

Steps 6–9: Compute Movement Capacities

Because of the presence of pedestrians, the computation steps provided earlier in this chapter should be used.

Step 6: Rank 1 Movement Capacity

The methodology assumes that Rank 1 vehicles are unimpeded by pedestrians.

Step 7: Rank 2 Movement Capacity

Step 7a: Pedestrian Impedance

The factor accounting for pedestrian blockage is computed by Equation 32-1 as follows:

$$f_{pb,13} = \frac{(v_{13})\left(\frac{w}{S_p}\right)}{3,600} = \frac{20\left(\frac{12}{3.5}\right)}{3,600} = 0.019$$

$$f_{pb,15} = \frac{(v_{15})\left(\frac{w}{S_p}\right)}{3,600} = \frac{20\left(\frac{12}{3.5}\right)}{3,600} = 0.019$$

The pedestrian impedance factor for each pedestrian movement x, $p_{p,x}$, is computed by Equation 32-2 as follows:

$$p_{p,13} = 1 - f_{pb,13} = 1 - 0.019 = 0.981$$

$$p_{p,15} = 1 - f_{pb,15} = 1 - 0.019 = 0.981$$

Step 7b: Movement Capacity for Major-Street Left-Turn Movements

On the basis of Exhibit 32-4, vehicular Movement 4 is impeded by pedestrian Movement 15. Therefore, the movement capacity for Rank 2 major-street left-turn movements is computed as follows:

$$c_{m,4} = (c_{p,4})p_{p,15} = (348)(0.981) = 341$$

Step 7c: Movement Capacity for Minor-Street Right-Turn Movements

The northbound minor-street right-turn movement (Movement 9) is impeded by one conflicting pedestrian movement: Movement 15.

$$f_9 = p_{p,15} = 0.981$$

The movement capacity is then computed as follows:

$$c_{m,9} = (c_{p,9})f_9 = (433)(0.981) = 425$$

Step 7d: Movement Capacity for Major-Street U-Turn Movements

The eastbound U-turn is unimpeded by queues from any other movement. Therefore, $f_{1U} = 1$, and the movement capacity is computed as follows:

$$c_{m,1U} = (c_{p,1U})f_{1U} = (523)(1) = 523$$

For the westbound U-turn, the movement capacity is found by first computing a capacity adjustment factor that accounts for the impeding effects of minor-street right turns as follows:

$$f_{4U} = p_{0,9} = 1 - \frac{v_9}{c_{m,9}} = 1 - \frac{100}{425} = 0.765$$

The movement capacity is therefore computed as follows:

$$c_{m,4U} = (c_{p,4U})f_{4U} = (629)(0.765) = 481$$

Because the westbound left-turn and U-turn movements are conducted from the same lane, their shared-lane capacity is computed as follows:

$$c_{m,4+4U} = \frac{v_4 + v_{4U}}{\frac{v_4}{c_{m,4}} + \frac{v_{4U}}{c_{m,4U}}} = \frac{100+25}{\frac{100}{341} + \frac{25}{481}} = 362$$

Step 7e: Effect of Major-Street Shared Through and Left-Turn Lane

This step is skipped.

Step 8: Compute Movement Capacities for Rank 3 Movements

There are no minor-street through movements, so the minor-street left-turn movement is treated as a Rank 3 movement.

Step 8a: Pedestrian Impedance

The northbound minor-street left turn (Movement 7) must yield to pedestrian Movements 13 and 15. Therefore, the impedance factor for pedestrians is as follows:

$$p_{p,7} = (p_{p,15})(p_{p,13}) = (0.981)(0.981) = 0.962$$

Step 8b: Rank 3 Movement Capacity for One-Stage Movements

The movement capacity $c_{m,k}$ for all Rank 3 movements is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements, assuming the movement operates in one stage. This value is computed as follows:

$$f_7 = (p_{0,1U})(p_{0,4+4U})p_{p,7} = \left(1 - \frac{v_{1U}}{c_{m,1U}}\right)\left(1 - \frac{v_{4+4U}}{c_{m,4+4U}}\right)p_{p,7}$$

$$f_7 = \left(1 - \frac{50}{523}\right)\left(1 - \frac{100+25}{362}\right)(0.962) = 0.570$$

$$c_{m,7} = (c_{p,7})f_7 = (112)(0.570) = 64$$

Step 8c: Rank 3 Capacity for Two-Stage Movements

Because the minor-street left-turn movement operates in two stages, the procedure for computing the total movement capacity for the subject movement considering the two-stage gap-acceptance process is followed.

First, the movement capacities for each stage of the left-turn movement are computed on the basis of the impeding movements for each stage. For Stage I, the left-turn movement is impeded by the major-street left and U-turns and by pedestrian Movement 15. Therefore:

$$f_{I,7} = (p_{0,1U})(p_{0,4+4U})p_{p,15} = \left(1 - \frac{v_{1U}}{c_{m,1U}}\right)\left(1 - \frac{v_{4+4U}}{c_{m,4+4U}}\right)p_{p,15}$$

$$f_{I,7} = \left(1 - \frac{50}{523}\right)\left(1 - \frac{100+25}{362}\right)(0.981) = 0.581$$

$$c_{m,I,7} = (c_{p,I,7})f_{I,7} = (207)(0.581) = 120$$

For Stage II, the left-turn movement is impeded only by pedestrian Movement 13. Therefore:

$$f_{II,7} = p_{p,13} = 0.981$$

$$c_{m,II,7} = (c_{p,II,7})f_{II,7} = (393)(0.981) = 386$$

Next, an adjustment factor *a* and an intermediate variable *y* are computed as follows:

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} = 1 - 0.32e^{-1.3\sqrt{1}} = 0.913$$

$$y_7 = \frac{c_{m,I,7} - c_{m,7}}{c_{m,II,7} - v_{4+4U} - c_{m,7}} = \frac{120 - 64}{386 - 125 - 64} = 0.284$$

Therefore, the total capacity c_T is computed as follows:

$$c_{T,7} = \frac{a}{{y_7}^{n_m+1} - 1}\left[y_7\left({y_7}^{n_m} - 1\right)\left(c_{m,II,7} - v_{4+4U}\right) + \left(y_7 - 1\right)c_{m,7}\right]$$

$$= \frac{0.913}{(0.284)^{1+1} - 1}\left[(0.284)\left((0.284)^1 - 1\right)(386 - 125) + (0.284 - 1)(64)\right] = 98$$

Step 9: Compute Movement Capacities for Rank 4 Movements

Because there are no Rank 4 movements, this step is skipped.

Step 10: Final Capacity Adjustments

There are no shared or flared lanes on the minor street, so this step is skipped.

Step 11: Compute Movement Control Delay

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The control delay for each minor movement is computed as follows:

$$d = \frac{3{,}600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5$$

$$d_{1U} = \frac{3{,}600}{523} + 900(0.25)\left[\frac{50}{523} - 1 + \sqrt{\left(\frac{50}{523} - 1\right)^2 + \frac{\left(\frac{3{,}600}{523}\right)\left(\frac{50}{523}\right)}{450(0.25)}}\right] + 5 = 12.6$$

This movement would be assigned LOS B.

$$d_{4+4U} = \frac{3{,}600}{362} + 900(0.25)\left[\frac{125}{362} - 1 + \sqrt{\left(\frac{125}{362} - 1\right)^2 + \frac{\left(\frac{3{,}600}{362}\right)\left(\frac{125}{362}\right)}{450(0.25)}}\right] + 5 = 20.1$$

This movement would be assigned LOS C.

$$d_9 = \frac{3{,}600}{425} + 900(0.25)\left[\frac{100}{425} - 1 + \sqrt{\left(\frac{100}{425} - 1\right)^2 + \frac{\left(\frac{3{,}600}{425}\right)\left(\frac{100}{425}\right)}{450(0.25)}}\right] + 5 = 16.1$$

This movement would be assigned LOS C.

$$d_7 = \frac{3{,}600}{98} + 900(0.25)\left[\frac{75}{98} - 1 + \sqrt{\left(\frac{75}{98} - 1\right)^2 + \frac{\left(\frac{3{,}600}{98}\right)\left(\frac{75}{98}\right)}{450(0.25)}}\right] + 5 = 113$$

This movement would be assigned LOS F.

Step 11b: Compute Control Delay to Rank 1 Movements

No shared lanes are present on the major street, so this step is skipped.

Step 12: Compute Approach and Intersection Control Delay

The control delay for each approach is computed as follows:

$$d_{A,EB} = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l} = \frac{(0)(100) + (0)(1{,}000) + (12.6)(50)}{100 + 1{,}000 + 50} = 0.5$$

$$d_{A,WB} = \frac{(0)(1{,}200) + (20.1)(125)}{1{,}200 + 125} = 1.9$$

$$d_{A,NB} = \frac{(16.1)(100) + (113)(75)}{100 + 75} = 57.6$$

The northbound approach is assigned LOS F. No LOS is assigned to the major-street approaches.

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB}}{v_{A,EB} + v_{A,WB} + v_{A,NB}}$$

$$= \frac{(0.5)(1{,}150) + (1.9)(1{,}325) + (57.6)(175)}{1{,}150 + 1{,}325 + 175} = 5.0$$

LOS is not defined for the intersection as a whole.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for each movement is computed as follows:

$$Q_{95} \approx 900T \left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}} \right)\left(\frac{v_x}{c_{m,x}} \right)}{150T}} \right] \left(\frac{c_{m,x}}{3{,}600} \right)$$

$$Q_{95,1U} \approx 900(0.25) \left[\frac{50}{523} - 1 + \sqrt{\left(\frac{50}{523} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{523} \right)\left(\frac{50}{523} \right)}{150(0.25)}} \right] \left(\frac{523}{3{,}600} \right) = 0.3$$

$$Q_{95,4+4U} \approx 900(0.25) \left[\frac{125}{362} - 1 + \sqrt{\left(\frac{125}{362} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{362} \right)\left(\frac{125}{362} \right)}{150(0.25)}} \right] \left(\frac{362}{3{,}600} \right) = 1.5$$

$$Q_{95,9} \approx 900(0.25) \left[\frac{100}{425} - 1 + \sqrt{\left(\frac{100}{425} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{425} \right)\left(\frac{100}{425} \right)}{150(0.25)}} \right] \left(\frac{425}{3{,}600} \right) = 0.9$$

$$Q_{95,7} \approx 900(0.25) \left[\frac{75}{98} - 1 + \sqrt{\left(\frac{75}{98} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{98} \right)\left(\frac{75}{98} \right)}{150(0.25)}} \right] \left(\frac{98}{3{,}600} \right) = 4.1$$

Discussion

Overall, the results indicate that, while most minor movements are operating at low to moderate delays and at LOS C or better, the minor-street left turn experiences high delays and operates at LOS F.

4. METHODOLOGY FOR THREE-LANE AWSC APPROACHES

This section provides details for analyzing three-lane approaches at all-way STOP-controlled intersections. Exhibit 32-16 provides the 512 possible combinations of probability of degree-of-conflict cases when alternative lane occupancies are considered for three-lane approaches. A 1 indicates that a vehicle is in the lane; a 0 indicates that a vehicle is not in the lane.

The probability adjustment is computed with Equation 32-9 through Equation 32-13 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined (assuming the 512 cases presented in Exhibit 32-16).

Equation 32-9
$$P(C_1) = P(1)$$

Equation 32-10
$$P(C_2) = \sum_{i=2}^{8} P(i)$$

Equation 32-11
$$P(C_3) = \sum_{i=9}^{22} P(i)$$

Equation 32-12
$$P(C_4) = \sum_{i=23}^{169} P(i)$$

Equation 32-13
$$P(C_5) = \sum_{i=170}^{512} P(i)$$

The probability adjustment factors are then computed with Equation 32-14 through Equation 32-18.

Equation 32-14
$$AdjP(1) = \alpha\left[P(C_2) + 2P(C_3) + 3P(C_4) + 4P(C_5)\right]/1$$

Equation 32-15
$$AdjP(2) \text{ through } AdjP(8) = \alpha\left[P(C_3) + 2P(C_4) + 3P(C_5) - P(C_2)\right]/7$$

Equation 32-16
$$AdjP(9) \text{ through } AdjP(22) = \alpha\left[P(C_4) + 2P(C_5) - 3P(C_3)\right]/14$$

Equation 32-17
$$AdjP(23) \text{ through } AdjP(169) = \alpha\left[P(C_5) - 6P(C_4)\right]/147$$

Equation 32-18
$$AdjP(170) \text{ through } AdjP(512) = -\alpha\left[10P(C_5)\right]/343$$

where α equals 0.01 (or 0.00 if correlation among saturation headways is not taken into account).

The remaining part of the procedure is fundamentally the same as for two-lane approaches.

Exhibit 32-16
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 1–49)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	**DOC Case**	**# of Vehicles**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**
1	1	0	0	0	0	0	0	0	0	0	0
2	2	1	1	0	0	0	0	0	0	0	0
3			0	1	0	0	0	0	0	0	0
4			0	0	1	0	0	0	0	0	0
5		2	1	1	0	0	0	0	0	0	0
6			0	1	1	0	0	0	0	0	0
7			1	0	1	0	0	0	0	0	0
8		3	1	1	1	0	0	0	0	0	0
9	3	1	0	0	0	1	0	0	0	0	0
10			0	0	0	0	1	0	0	0	0
11			0	0	0	0	0	1	0	0	0
12			0	0	0	0	0	0	1	0	0
13			0	0	0	0	0	0	0	1	0
14			0	0	0	0	0	0	0	0	1
15		2	0	0	0	1	1	0	0	0	0
16			0	0	0	0	1	1	0	0	0
17			0	0	0	1	0	1	0	0	0
18			0	0	0	0	0	0	1	1	0
19			0	0	0	0	0	0	0	1	1
20			0	0	0	0	0	0	1	0	1
21		3	0	0	0	1	1	1	0	0	0
22			0	0	0	0	0	0	1	1	1
23	4	2	1	0	0	1	0	0	0	0	0
24			1	0	0	0	1	0	0	0	0
25			1	0	0	0	0	1	0	0	0
26			0	1	0	1	0	0	0	0	0
27			0	1	0	0	1	0	0	0	0
28			0	1	0	0	0	1	0	0	0
29			0	0	1	1	0	0	0	0	0
30			0	0	1	0	1	0	0	0	0
31			0	0	1	0	0	1	0	0	0
32			1	0	0	0	0	0	1	0	0
33			1	0	0	0	0	0	0	1	0
34			1	0	0	0	0	0	0	0	1
35			0	1	0	0	0	0	1	0	0
36			0	1	0	0	0	0	0	1	0
37			0	1	0	0	0	0	0	0	1
38			0	0	1	0	0	0	1	0	0
39			0	0	1	0	0	0	0	1	0
40			0	0	1	0	0	0	0	0	1
41			0	0	0	1	0	0	1	0	0
42			0	0	0	1	0	0	0	1	0
43			0	0	0	1	0	0	0	0	1
44			0	0	0	0	1	0	1	0	0
45			0	0	0	0	1	0	0	1	0
46			0	0	0	0	1	0	0	0	1
47			0	0	0	0	0	1	1	0	0
48			0	0	0	0	0	1	0	1	0
49			0	0	0	0	0	1	0	0	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 50–112)

	DOC	# of	Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	Case	Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
50	4	3	1	1	0	1	0	0	0	0	0
51	(cont'd.)		1	1	0	0	1	0	0	0	0
52			1	1	0	0	0	1	0	0	0
53			0	1	1	1	0	0	0	0	0
54			0	1	1	0	1	0	0	0	0
55			0	1	1	0	0	1	0	0	0
56			1	0	1	1	0	0	0	0	0
57			1	0	1	0	1	0	0	0	0
58			1	0	1	0	0	1	0	0	0
59			1	1	0	0	0	0	1	0	0
60			1	1	0	0	0	0	0	1	0
61			1	1	0	0	0	0	0	0	1
62			0	1	1	0	0	0	1	0	0
63			0	1	1	0	0	0	0	1	0
64			0	1	1	0	0	0	0	0	1
65			1	0	1	0	0	0	1	0	0
66			1	0	1	0	0	0	0	1	0
67			1	0	1	0	0	0	0	0	1
68			1	0	0	1	1	0	0	0	0
69			1	0	0	0	1	1	0	0	0
70			1	0	0	1	0	1	0	0	0
71			0	1	0	1	1	0	0	0	0
72			0	1	0	0	1	1	0	0	0
73			0	1	0	1	0	1	0	0	0
74			0	0	1	1	1	0	0	0	0
75			0	0	1	0	1	1	0	0	0
76			0	0	1	1	0	1	0	0	0
77			0	0	0	1	1	0	1	0	0
78			0	0	0	1	1	0	0	1	0
79			0	0	0	1	1	0	0	0	1
80			0	0	0	0	1	1	1	0	0
81			0	0	0	0	1	1	0	1	0
82			0	0	0	0	1	1	0	0	1
83			0	0	0	1	0	1	1	0	0
84			0	0	0	1	0	1	0	1	0
85			0	0	0	1	0	1	0	0	1
86			1	0	0	0	0	0	1	1	0
87			1	0	0	0	0	0	0	1	1
88			1	0	0	0	0	0	1	0	1
89			0	1	0	0	0	0	1	1	0
90			0	1	0	0	0	0	0	1	1
91			0	1	0	0	0	0	1	0	1
92			0	0	1	0	0	0	1	1	0
93			0	0	1	0	0	0	0	1	1
94			0	0	1	0	0	0	1	0	1
95			0	0	0	1	0	0	1	1	0
96			0	0	0	1	0	0	0	1	1
97			0	0	0	1	0	0	1	0	1
98			0	0	0	0	1	0	1	1	0
99			0	0	0	0	1	0	0	1	1
100			0	0	0	0	1	0	1	0	1
101			0	0	0	0	0	1	1	1	0
102			0	0	0	0	0	1	0	1	1
103			0	0	0	0	0	1	1	0	1
104		4	1	1	0	1	1	0	0	0	0
105			1	1	0	0	1	1	0	0	0
106			1	1	0	1	0	1	0	0	0
107			0	1	1	1	1	0	0	0	0
108			0	1	1	0	1	1	0	0	0
109			0	1	1	1	0	1	0	0	0
110			1	0	1	1	1	0	0	0	0
111			1	0	1	0	1	1	0	0	0
112			1	0	1	1	0	1	0	0	0

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 113–175)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	**DOC Case**	**# of Vehicles**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**
113	4	4	1	1	0	0	0	0	1	1	0
114	(cont'd.)	(cont'd.)	1	1	0	0	0	0	0	1	1
115			1	1	0	0	0	0	1	0	1
116			0	1	1	0	0	0	1	1	0
117			0	1	1	0	0	0	0	1	1
118			0	1	1	0	0	0	1	0	1
119			1	0	1	0	0	0	1	1	0
120			1	0	1	0	0	0	0	1	1
121			1	0	1	0	0	0	1	0	1
122			0	0	0	1	1	0	1	1	0
123			0	0	0	1	1	0	0	1	1
124			0	0	0	1	1	0	1	0	1
125			0	0	0	0	1	1	1	1	0
126			0	0	0	0	1	1	0	1	1
127			0	0	0	0	1	1	1	0	1
128			0	0	0	1	0	1	1	1	0
129			0	0	0	1	0	1	0	1	1
130			0	0	0	1	0	1	1	0	1
131			1	1	1	1	0	0	0	0	0
132			1	1	1	0	1	0	0	0	0
133			1	1	1	0	0	1	0	0	0
134			1	1	1	0	0	0	1	0	0
135			1	1	1	0	0	0	0	1	0
136			1	1	1	0	0	0	0	0	1
137			1	0	0	1	1	1	0	0	0
138			0	1	0	1	1	1	0	0	0
139			0	0	1	1	1	1	0	0	0
140			0	0	0	1	1	1	1	0	0
141			0	0	0	1	1	1	0	1	0
142			0	0	0	1	1	1	0	0	1
143			1	0	0	0	0	0	1	1	1
144			0	1	0	0	0	0	1	1	1
145			0	0	1	0	0	0	1	1	1
146			0	0	0	1	0	0	1	1	1
147			0	0	0	0	1	0	1	1	1
148			0	0	0	0	0	1	1	1	1
149		5	1	1	1	1	1	0	0	0	0
150			1	1	1	0	1	1	0	0	0
151			1	1	1	1	0	1	0	0	0
152			1	1	1	0	0	0	1	1	0
153			1	1	1	0	0	0	0	1	1
154			1	1	1	0	0	0	1	0	1
155			1	1	0	1	1	1	0	0	0
156			0	1	1	1	1	1	0	0	0
157			1	0	1	1	1	1	0	0	0
158			0	0	0	1	1	1	1	1	0
159			0	0	0	1	1	1	0	1	1
160			0	0	0	1	1	1	1	0	1
161			1	1	0	0	0	0	1	1	1
162			0	1	1	0	0	0	1	1	1
163			1	0	1	0	0	0	1	1	1
164			0	0	0	1	1	0	1	1	1
165			0	0	0	0	1	1	1	1	1
166			0	0	0	1	0	1	1	1	1
167		6	1	1	1	1	1	1	0	0	0
168			1	1	1	0	0	0	1	1	1
169			0	0	0	1	1	1	1	1	1
170	5	3	1	0	0	1	0	0	1	0	0
171			1	0	0	1	0	0	0	1	0
172			1	0	0	1	0	0	0	0	1
173			1	0	0	0	1	0	1	0	0
174			1	0	0	0	1	0	0	1	0
175			1	0	0	0	1	0	0	0	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 176–238)

i	DOC Case	# of Vehicles	Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
			L1	L2	L3	L1	L2	L3	L1	L2	L3
176	5	3	1	0	0	0	0	1	1	0	0
177	(cont'd.)	(cont'd.)	1	0	0	0	0	1	0	1	0
178			1	0	0	0	0	1	0	0	1
179			0	1	0	1	0	0	1	0	0
180			0	1	0	1	0	0	0	1	0
181			0	1	0	1	0	0	0	0	1
182			0	1	0	0	1	0	1	0	0
183			0	1	0	0	1	0	0	1	0
184			0	1	0	0	1	0	0	0	1
185			0	1	0	0	0	1	1	0	0
186			0	1	0	0	0	1	0	1	0
187			0	1	0	0	0	1	0	0	1
188			0	0	1	1	0	0	1	0	0
189			0	0	1	1	0	0	0	1	0
190			0	0	1	1	0	0	0	0	1
191			0	0	1	0	1	0	1	0	0
192			0	0	1	0	1	0	0	1	0
193			0	0	1	0	1	0	0	0	1
194			0	0	1	0	0	1	1	0	0
195			0	0	1	0	0	1	0	1	0
196			0	0	1	0	0	1	0	0	1
197		4	1	1	0	1	0	0	1	0	0
198			1	1	0	1	0	0	0	1	0
199			1	1	0	1	0	0	0	0	1
200			1	1	0	0	1	0	1	0	0
201			1	1	0	0	1	0	0	1	0
202			1	1	0	0	1	0	0	0	1
203			1	1	0	0	0	1	1	0	0
204			1	1	0	0	0	1	0	1	0
205			1	1	0	0	0	1	0	0	1
206			0	1	1	1	0	0	1	0	0
207			0	1	1	1	0	0	0	1	0
208			0	1	1	1	0	0	0	0	1
209			0	1	1	0	1	0	1	0	0
210			0	1	1	0	1	0	0	1	0
211			0	1	1	0	1	0	0	0	1
212			0	1	1	0	0	1	1	0	0
213			0	1	1	0	0	1	0	1	0
214			0	1	1	0	0	1	0	0	1
215			1	0	1	1	0	0	1	0	0
216			1	0	1	1	0	0	0	1	0
217			1	0	1	1	0	0	0	0	1
218			1	0	1	0	1	0	1	0	0
219			1	0	1	0	1	0	0	1	0
220			1	0	1	0	1	0	0	0	1
221			1	0	1	0	0	1	1	0	0
222			1	0	1	0	0	1	0	1	0
223			1	0	1	0	0	1	0	0	1
224			1	0	0	1	1	0	1	0	0
225			1	0	0	1	1	0	0	1	0
226			1	0	0	1	1	0	0	0	1
227			1	0	0	0	1	1	1	0	0
228			1	0	0	0	1	1	0	1	0
229			1	0	0	0	1	1	0	0	1
230			1	0	0	1	0	1	1	0	0
231			1	0	0	1	0	1	0	1	0
232			1	0	0	1	0	1	0	0	1
233			0	1	0	1	1	0	1	0	0
234			0	1	0	1	1	0	0	1	0
235			0	1	0	1	1	0	0	0	1
236			0	1	0	0	1	1	1	0	0
237			0	1	0	0	1	1	0	1	0
238			0	1	0	0	1	1	0	0	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 239–301)

i	DOC Case	# of Vehicles	Opposing Approach L1	L2	L3	Conflicting Left Approach L1	L2	L3	Conflicting Right Approach L1	L2	L3
239	5	4	0	1	0	1	0	1	1	0	0
240	(cont'd.)	(cont'd.)	0	1	0	1	0	1	0	1	0
241			0	1	0	1	0	1	0	0	1
242			0	0	1	1	1	0	1	0	0
243			0	0	1	1	1	0	0	1	0
244			0	0	1	1	1	0	0	0	1
245			0	0	1	0	1	1	1	0	0
246			0	0	1	0	1	1	0	1	0
247			0	0	1	0	1	1	0	0	1
248			0	0	1	1	0	1	1	0	0
249			0	0	1	1	0	1	0	1	0
250			0	0	1	1	0	1	0	0	1
251			1	0	0	1	0	0	1	1	0
252			1	0	0	1	0	0	0	1	1
253			1	0	0	1	0	0	1	0	1
254			1	0	0	0	1	0	1	1	0
255			1	0	0	0	1	0	0	1	1
256			1	0	0	0	1	0	1	0	1
257			1	0	0	0	0	1	1	1	0
258			1	0	0	0	0	1	0	1	1
259			1	0	0	0	0	1	1	0	1
260			0	1	0	1	0	0	1	1	0
261			0	1	0	1	0	0	0	1	1
262			0	1	0	1	0	0	1	0	1
263			0	1	0	0	1	0	1	1	0
264			0	1	0	0	1	0	0	1	1
265			0	1	0	0	1	0	1	0	1
266			0	1	0	0	0	1	1	1	0
267			0	1	0	0	0	1	0	1	1
268			0	1	0	0	0	1	1	0	1
269			0	0	1	1	0	0	1	1	0
270			0	0	1	1	0	0	0	1	1
271			0	0	1	1	0	0	1	0	1
272			0	0	1	0	1	0	1	1	0
273			0	0	1	0	1	0	0	1	1
274			0	0	1	0	1	0	1	0	1
275			0	0	1	0	0	1	1	1	0
276			0	0	1	0	0	1	0	1	1
277			0	0	1	0	0	1	1	0	1
278		5	1	1	0	1	1	0	1	0	0
279			1	1	0	1	1	0	0	1	0
280			1	1	0	1	1	0	0	0	1
281			1	1	0	0	1	1	1	0	0
282			1	1	0	0	1	1	0	1	0
283			1	1	0	0	1	1	0	0	1
284			1	1	0	1	0	1	1	0	0
285			1	1	0	1	0	1	0	1	0
286			1	1	0	1	0	1	0	0	1
287			0	1	1	1	1	0	1	0	0
288			0	1	1	1	1	0	0	1	0
289			0	1	1	1	1	0	0	0	1
290			0	1	1	0	1	1	1	0	0
291			0	1	1	0	1	1	0	1	0
292			0	1	1	0	1	1	0	0	1
293			0	1	1	1	0	1	1	0	0
294			0	1	1	1	0	1	0	1	0
295			0	1	1	1	0	1	0	0	1
296			1	0	1	1	1	0	1	0	0
297			1	0	1	1	1	0	0	1	0
298			1	0	1	1	1	0	0	0	1
299			1	0	1	0	1	1	1	0	0
300			1	0	1	0	1	1	0	1	0
301			1	0	1	0	1	1	0	0	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 302–364)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	**DOC Case**	**# of Vehicles**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**
302	5	5	1	0	1	1	0	1	1	0	0
303	(cont'd.)	(cont'd.)	1	0	1	1	0	1	0	1	0
304			1	0	1	1	0	1	0	0	1
305			1	1	0	1	0	0	1	1	0
306			1	1	0	0	1	0	1	1	0
307			1	1	0	0	0	1	1	1	0
308			1	1	0	1	0	0	0	1	1
309			1	1	0	0	1	0	0	1	1
310			1	1	0	0	0	1	0	1	1
311			1	1	0	1	0	0	1	0	1
312			1	1	0	0	1	0	1	0	1
313			1	1	0	0	0	1	1	0	1
314			0	1	1	1	0	0	1	1	0
315			0	1	1	0	1	0	1	1	0
316			0	1	1	0	0	1	1	1	0
317			0	1	1	1	0	0	0	1	1
318			0	1	1	0	1	0	0	1	1
319			0	1	1	0	0	1	0	1	1
320			0	1	1	1	0	0	1	0	1
321			0	1	1	0	1	0	1	0	1
322			0	1	1	0	0	1	1	0	1
323			1	0	1	1	0	0	1	1	0
324			1	0	1	0	1	0	1	1	0
325			1	0	1	0	0	1	1	1	0
326			1	0	1	1	0	0	0	1	1
327			1	0	1	0	1	0	0	1	1
328			1	0	1	0	0	1	0	1	1
329			1	0	1	1	0	0	1	0	1
330			1	0	1	0	1	0	1	0	1
331			1	0	1	0	0	1	1	0	1
332			1	0	0	1	1	0	1	1	0
333			1	0	0	1	1	0	0	1	1
334			1	0	0	1	1	0	1	0	1
335			1	0	0	0	1	1	1	1	0
336			1	0	0	0	1	1	0	1	1
337			1	0	0	0	1	1	1	0	1
338			1	0	0	1	0	1	1	1	0
339			1	0	0	1	0	1	0	1	1
340			1	0	0	1	0	1	1	0	1
341			0	1	0	1	1	0	1	1	0
342			0	1	0	1	1	0	0	1	1
343			0	1	0	1	1	0	1	0	1
344			0	1	0	0	1	1	1	1	0
345			0	1	0	0	1	1	0	1	1
346			0	1	0	0	1	1	1	0	1
347			0	1	0	1	0	1	1	1	0
348			0	1	0	1	0	1	0	1	1
349			0	1	0	1	0	1	1	0	1
350			0	0	1	1	1	0	1	1	0
351			0	0	1	1	1	0	0	1	1
352			0	0	1	1	1	0	1	0	1
353			0	0	1	0	1	1	1	1	0
354			0	0	1	0	1	1	0	1	1
355			0	0	1	0	1	1	1	0	1
356			0	0	1	1	0	1	1	1	0
357			0	0	1	1	0	1	0	1	1
358			0	0	1	1	0	1	1	0	1
359			1	1	1	1	0	0	1	0	0
360			1	1	1	1	0	0	0	1	0
361			1	1	1	1	0	0	0	0	1
362			1	1	1	0	1	0	1	0	0
363			1	1	1	0	1	0	0	1	0
364			1	1	1	0	1	0	0	0	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 365–427)

i	DOC Case	# of Vehicles	Opposing Approach L1	Opposing Approach L2	Opposing Approach L3	Conflicting Left Approach L1	Conflicting Left Approach L2	Conflicting Left Approach L3	Conflicting Right Approach L1	Conflicting Right Approach L2	Conflicting Right Approach L3
365	5	5	1	1	1	0	0	1	1	0	0
366	(cont'd.	(cont'd.)	1	1	1	0	0	1	0	1	0
367			1	1	1	0	0	1	0	0	1
368			1	0	0	1	1	1	1	0	0
369			1	0	0	1	1	1	0	1	0
370			1	0	0	1	1	1	0	0	1
371			0	1	0	1	1	1	1	0	0
372			0	1	0	1	1	1	0	1	0
373			0	1	0	1	1	1	0	0	1
374			0	0	1	1	1	1	1	0	0
375			0	0	1	1	1	1	0	1	0
376			0	0	1	1	1	1	0	0	1
377			1	0	0	1	0	0	1	1	1
378			1	0	0	0	1	0	1	1	1
379			1	0	0	0	0	1	1	1	1
380			0	1	0	1	0	0	1	1	1
381			0	1	0	0	1	0	1	1	1
382			0	1	0	0	0	1	1	1	1
383			0	0	1	1	0	0	1	1	1
384			0	0	1	0	1	0	1	1	1
385			0	0	1	0	0	1	1	1	1
386		6	1	1	0	1	1	0	1	1	0
387			1	1	0	1	1	0	0	1	1
388			1	1	0	1	1	0	1	0	1
389			1	1	0	0	1	1	1	1	0
390			1	1	0	0	1	1	0	1	1
391			1	1	0	0	1	1	1	0	1
392			1	1	0	1	0	1	1	1	0
393			1	1	0	1	0	1	0	1	1
394			1	1	0	1	0	1	1	0	1
395			0	1	1	1	1	0	1	1	0
396			0	1	1	1	1	0	0	1	1
397			0	1	1	1	1	0	1	0	1
398			0	1	1	0	1	1	1	1	0
399			0	1	1	0	1	1	0	1	1
400			0	1	1	0	1	1	1	0	1
401			0	1	1	1	0	1	1	1	0
402			0	1	1	1	0	1	0	1	1
403			0	1	1	1	0	1	1	0	1
404			1	0	1	1	1	0	1	1	0
405			1	0	1	1	1	0	0	1	1
406			1	0	1	1	1	0	1	0	1
407			1	0	1	0	1	1	1	1	0
408			1	0	1	0	1	1	0	1	1
409			1	0	1	0	1	1	1	0	1
410			1	0	1	1	0	1	1	1	0
411			1	0	1	1	0	1	0	1	1
412			1	0	1	1	0	1	1	0	1
413			1	1	1	1	1	0	1	0	0
414			1	1	1	1	1	0	0	1	0
415			1	1	1	1	1	0	0	0	1
416			1	1	1	0	1	1	1	0	0
417			1	1	1	0	1	1	0	1	0
418			1	1	1	0	1	1	0	0	1
419			1	1	1	1	0	1	1	0	0
420			1	1	1	1	0	1	0	1	0
421			1	1	1	1	0	1	0	0	1
422			1	1	1	1	0	0	1	1	0
423			1	1	1	1	0	0	0	1	1
424			1	1	1	1	0	0	1	0	1
425			1	1	1	0	1	0	1	1	0
426			1	1	1	0	1	0	0	1	1
427			1	1	1	0	1	0	1	0	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 428–490)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	# of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
428	5	6	1	1	1	0	0	1	1	1	0
429	(cont'd.)	(cont'd.)	1	1	1	0	0	1	0	1	1
430			1	1	1	0	0	1	1	0	1
431			1	1	0	1	1	1	1	0	0
432			1	1	0	1	1	1	0	1	0
433			1	1	0	1	1	1	0	0	1
434			0	1	1	1	1	1	1	0	0
435			0	1	1	1	1	1	0	1	0
436			0	1	1	1	1	1	0	0	1
437			1	0	1	1	1	1	1	0	0
438			1	0	1	1	1	1	0	1	0
439			1	0	1	1	1	1	0	0	1
440			1	0	0	1	1	1	1	1	0
441			1	0	0	1	1	1	0	1	1
442			1	0	0	1	1	1	1	0	1
443			0	1	0	1	1	1	1	1	0
444			0	1	0	1	1	1	0	1	1
445			0	1	0	1	1	1	1	0	1
446			0	0	1	1	1	1	1	1	0
447			0	0	1	1	1	1	0	1	1
448			0	0	1	1	1	1	1	0	1
449			1	1	0	1	0	0	1	1	1
450			1	1	0	0	1	0	1	1	1
451			1	1	0	0	0	1	1	1	1
452			0	1	1	1	0	0	1	1	1
453			0	1	1	0	1	0	1	1	1
454			0	1	1	0	0	1	1	1	1
455			1	0	1	1	0	0	1	1	1
456			1	0	1	0	1	0	1	1	1
457			1	0	1	0	0	1	1	1	1
458			1	0	0	1	1	0	1	1	1
459			1	0	0	0	1	1	1	1	1
460			1	0	0	1	0	1	1	1	1
461			0	1	0	1	1	0	1	1	1
462			0	1	0	0	1	1	1	1	1
463			0	1	0	1	0	1	1	1	1
464			0	0	1	1	1	0	1	1	1
465			0	0	1	0	1	1	1	1	1
466			0	0	1	1	0	1	1	1	1
467		7	1	1	1	1	1	0	1	1	0
468			1	1	1	1	1	0	0	1	1
469			1	1	1	1	1	0	1	0	1
470			1	1	1	0	1	1	1	1	0
471			1	1	1	0	1	1	0	1	1
472			1	1	1	0	1	1	1	0	1
473			1	1	1	1	0	1	1	1	0
474			1	1	1	1	0	1	0	1	1
475			1	1	1	1	0	1	1	0	1
476			1	1	0	1	1	1	1	1	0
477			1	1	0	1	1	1	0	1	1
478			1	1	0	1	1	1	1	0	1
479			0	1	1	1	1	1	1	1	0
480			0	1	1	1	1	1	0	1	1
481			0	1	1	1	1	1	1	0	1
482			1	0	1	1	1	1	1	1	0
483			1	0	1	1	1	1	0	1	1
484			1	0	1	1	1	1	1	0	1
485			1	1	0	1	1	0	1	1	1
486			1	1	0	0	1	1	1	1	1
487			1	1	0	1	0	1	1	1	1
488			0	1	1	1	1	0	1	1	1
489			0	1	1	0	1	1	1	1	1
490			0	1	1	1	0	1	1	1	1

Exhibit 32-16 (cont'd.)
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 491–512)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	# of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
491	5	7	1	0	1	1	1	0	1	1	1
492	(cont'd.)	(cont'd.)	1	0	1	0	1	1	1	1	1
493			1	0	1	1	0	1	1	1	1
494			1	1	1	1	1	1	1	0	0
495			1	1	1	1	1	1	0	1	0
496			1	1	1	1	1	1	0	0	1
497			1	1	1	1	0	0	1	1	1
498			1	1	1	0	1	0	1	1	1
499			1	1	1	0	0	1	1	1	1
500			1	0	0	1	1	1	1	1	1
501			0	1	0	1	1	1	1	1	1
502			0	0	1	1	1	1	1	1	1
503		8	1	1	1	1	1	1	1	1	0
504			1	1	1	1	1	1	0	1	1
505			1	1	1	1	1	1	1	0	1
506			1	1	1	1	1	0	1	1	1
507			1	1	1	0	1	1	1	1	1
508			1	1	1	1	0	1	1	1	1
509			1	1	0	1	1	1	1	1	1
510			0	1	1	1	1	1	1	1	1
511			1	0	1	1	1	1	1	1	1
512		9	1	1	1	1	1	1	1	1	1

Note: "DOC Case" is the degree-of-conflict case, "# of vehicles" is the total number of vehicles on the opposing and conflicting approaches, L1 is Lane 1, L2 is Lane 2, and L3 is Lane 3.

5. AWSC SUPPLEMENTAL EXAMPLE PROBLEMS

Example Problem 1 appears in Chapter 20.

This part of the chapter provides additional example problems for use of the AWSC methodology. Exhibit 32-17 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operations analysis level in terms of the calculations, except that default values are used where available.

Exhibit 32-17
AWSC Example Problems

Problem Number	Description	Analysis Level
1	Details for Example Problem 1 (Chapter 20)	Operational
2	Analyze a multilane, four-leg AWSC intersection	Operational

HEADWAY ADJUSTMENT FACTOR CALCULATION DETAILS FOR AWSC EXAMPLE PROBLEM 1

This section provides details of the calculations used to determine the headway adjustment factors in Example Problem 1 in Chapter 20, All-Way STOP-Controlled Intersections. In the following, EB means eastbound, WB means westbound, NB means northbound, SB means southbound, L1 means Lane 1, and L2 means Lane 2.

	EB L1	EB L2	WB L1	WB L2	NB L1	NB L2	SB L1	SB L2
Total lane flow rate	368		421				158	
LT flow rate in lane	53	0	0	0	0	0	105	0
RT flow rate in lane	0	0	105	0	0	0	53	0
Prop LT in lane	0.143		0.000				0.667	
Prop RT in lane	0.000		0.250				0.333	
Prop HV in Lane	0.02		0.02				0.02	
Geometry Group	1		1				1	
hL Tadj T 10-18	0.2		0.2				0.2	
hR Tadj T 10-18	-0.6		-0.6				-0.6	
hH Tadj T 10-18	1.7		1.7				1.7	
hadj	0.063		-0.116				-0.033	

	EB L1	EB L2	WB L1	WB L2	NB L1	NB L2	SB L1	SB L2
Total lane flow rate	368		421				158	
hd, initial value, Iteration 1	3.2		3.2				3.2	
x, initial, Iteration 1	0.327		0.374				0.140	
hd, computed value, Iteration 1	4.57		4.35				5.14	
Convergence?	N		N				N	
hd, initial value, Iteration 2	4.57		4.35				5.14	
x, initial, Iteration 2	0.468		0.509				0.225	
hd, computed value, Iteration 2	4.88		4.66				5.59	
Convergence?	N		N				N	
hd, initial value, Iteration 3	4.88		4.66				5.59	
x, initial, Iteration 3	0.499		0.545				0.245	
hd, computed value, Iteration 3	4.95		4.73				5.70	
Convergence?	Y		Y				N	
hd, initial value, Iteration 4	4.88		4.66				5.70	
x, initial, Iteration 4	0.499		0.545				0.250	
hd, computed value, Iteration 4	4.97		4.74				5.70	
Convergence?	Y		Y				Y	

Eastbound approach (EB L1), Iteration 1

	PaO1	PaCL1	PaCR1	P[Ci]	h_base	h_adj	h_si
1	0.6257	0.8596	1	0.5379	3.900	0.063	3.963
2	0.3743	0.8596	1	0.3217	4.700	0.063	4.763
5	0.6257	0.1404	1	0.0878	5.800	0.063	5.863
7	0.6257	0.8596	0	0	5.800		
13	0.6257	0.1404	0	0	7.000		
16	0.3743	0.1404	1	0.0525	7.000	0.063	7.063
21	0.3743	0.8596	0	0	7.000		
45	0.3743	0.1404	0	0	9.600		
						Sum:	4.571

DOC	SumPC1	SumPC2	SumPC3	SumPC4	SumPC5			Pcadj	PCiFinal
1	0.53791							0.006550	0.544459
2		0.32174						-0.000430	0.321310
3			0.08782					-0.000352	0.087470
3			0.00000					-0.000352	-0.000352
4				0.00000				-0.000117	-0.000117
4				0.05253				-0.000117	0.052412
4				0.00000				-0.000117	-0.000117
5					0.00000			0.000000	0.000000
PC	0.53791	0.32174	0.08782	0.05253	0.00000				
Count	1	3	6	27	27				
PCAdj	0.00655	-0.00043	-0.00035	-0.00012	0.00000	0.0057	Sum:	0.000000	1.000000

	EB L1	EB L2	WB L1	WB L2	NB L1	NB L2	SB L1	SB L2
Total lane flow rate	368		421				158	
Service time	2.97		2.74				3.70	
Degree of utilization, x	0.508		0.555				0.250	
Departure headway, hd	4.97		4.74				5.70	
Capacity								
Delay	13.0		13.5				10.6	
Level of service	B		B				B	
Delay, approach	13.0		13.5				10.6	
LOS approach	B		B				B	
Delay, intersection				12.8				
LOS intersection				B				
95th percentile queue length	2.9		3.5				1.0	

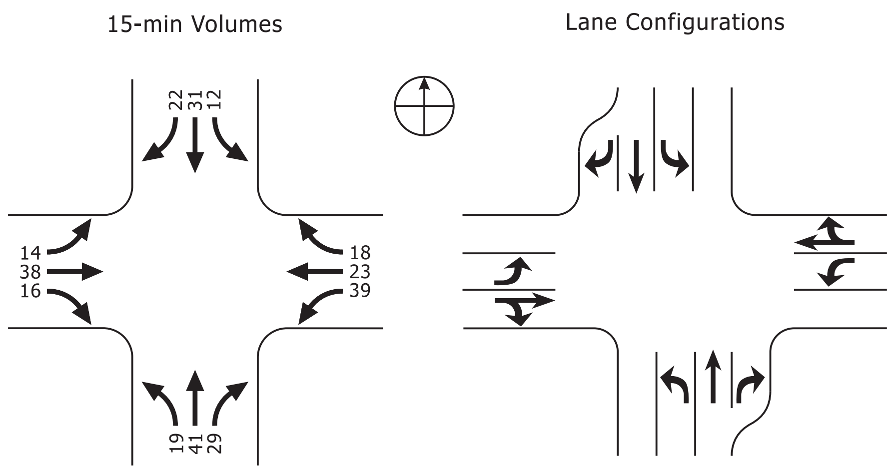
AWSC EXAMPLE PROBLEM 2: MULTILANE, FOUR-LEG INTERSECTION

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Four legs;
- Two-lane approaches on the east and west legs;
- Three-lane approaches on the north and south legs;
- Percent heavy vehicles on all approaches = 2%;
- Demand volumes are provided in 15-min intervals (therefore, a peak hour factor is not required), and the analysis period length is 0.25 h; and
- Volumes and lane configurations as shown in Exhibit 32-18.

Exhibit 32-18
AWSC Example Problem 2: 15-min Volumes and Lane Configurations



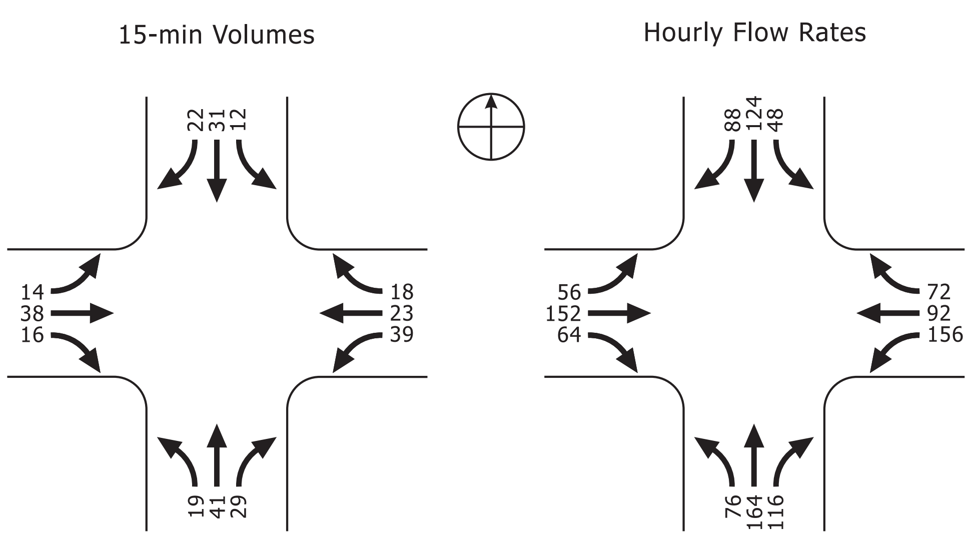
Comments

All input parameters are known, so no default values are needed or used. The use of a spreadsheet or software is required because of the several thousand repetitive computations needed. Slight differences in reported values may result from rounding differences between manual and software computations. Because it is not practical to show all the individual computations, this example problem will show how one or more computations are made. All the computational results can be found in the spreadsheet output, located in the Volume 4 Technical Reference Library for Chapter 20, All-Way STOP-Controlled Intersections.

Step 1: Convert Movement Demand Volumes to Flow Rates

To convert the peak 15-min demand volumes to hourly flow rates, the individual movement volumes are simply multiplied by four, as shown in Exhibit 32-19:

Exhibit 32-19
AWSC Example Problem 2: Hourly Flow Rates



Step 2: Determine Lane Flow Rates

This step simply involves assigning the turning movement volume to each of the approach lanes. The left-turn volume is assigned to the separate left-turn lane on each approach. For the east and west approaches, the through and right-turn

volumes are assigned to the shared through and right lanes. For the north and south approaches, the through volumes are assigned to the through lanes and the right-turn volumes are assigned to the right-turn lanes.

Step 3: Determine Geometry Group for Each Approach

Exhibit 20-10 shows that each approach should be assigned to Geometry Group 6.

Step 4: Determine Saturation Headway Adjustments

Exhibit 20-11 shows that the headway adjustments for left turns, right turns, and heavy vehicles are 0.5, –0.7, and 1.7, respectively. These values apply to all approaches as all are assigned Geometry Group 6. The saturation headway adjustment for the eastbound approach is as follows for Lane 1 (the left-turn lane):

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

$$h_{adj} = 0.5(1.0) - 0.7(0) + 1.7(0.02) = 0.534$$

Similarly, the saturation headway adjustment for Lane 2 of the eastbound approach is as follows:

$$h_{adj} = 0.2(0) - 0.6\frac{64}{64+216} + 1.7(0.02) = -0.173$$

The saturation headway adjustment for all the remaining lanes by approach is similarly calculated. The full computational results can be seen in the "HdwyAdj" spreadsheet tab.

Steps 5 Through 11: Determine Departure Headway

These steps are iterative and, for this example, involve several thousand calculations. The following narrative highlights some of the key calculations using the eastbound approach for Iteration 1 but does not attempt to reproduce all calculations for all iterations. The full computational results for each of the iterative computations can be seen in the "DepHdwyIterX" spreadsheet tab, where "X" is the iteration.

Step 6: Calculate Initial Degree of Utilization

The remainder of this example illustrates the calculations needed to evaluate Lane 1 on the eastbound approach (eastbound left turn). Step 6 requires calculating the initial degree of utilization for all the opposing and conflicting lanes. They are computed as follows:

$$x, WB(\text{Lane }1) = \frac{vh_d}{3{,}600} = \frac{(156)(3.2)}{3{,}600} = 0.1387$$

$$x, WB(\text{Lane }2) = \frac{vh_d}{3{,}600} = \frac{(164)(3.2)}{3{,}600} = 0.1458$$

$$x, NB(\text{Lane }1) = \frac{vh_d}{3{,}600} = \frac{(76)(3.2)}{3{,}600} = 0.0676$$

$$x, NB(\text{Lane 2}) = \frac{vh_d}{3{,}600} = \frac{(164)(3.2)}{3{,}600} = 0.1458$$

$$x, NB(\text{Lane 3}) = \frac{vh_d}{3{,}600} = \frac{(116)(3.2)}{3{,}600} = 0.1031$$

$$x, SB(\text{Lane 1}) = \frac{vh_d}{3{,}600} = \frac{(48)(3.2)}{3{,}600} = 0.0427$$

$$x, SB(\text{Lane 2}) = \frac{vh_d}{3{,}600} = \frac{(124)(3.2)}{3{,}600} = 0.1102$$

$$x, SB(\text{Lane 3}) = \frac{vh_d}{3{,}600} = \frac{(88)(3.2)}{3{,}600} = 0.0782$$

Step 7: Compute Probability States

The probability state of each combination *i* is determined with Equation 20-15:

$$P(i) = \prod_j P(a_j) = P(a_O)P(a_{CL})P(a_{CR})$$

As illustrated in Example Problem 1 in Chapter 20, for an intersection with single-lane approaches, only eight cases from Exhibit 20-13 apply. However, to evaluate three-lane approaches, Exhibit 32-16 is required. Exhibit 32-16 is an expanded version of Exhibit 20-13 that provides the 512 possible combinations of probability of degree-of-conflict cases.

For example, the probability state for the eastbound leg under the condition of no opposing vehicles on the other approaches (Degree-of-Conflict Case 1, $i = 1$) is as follows (using Exhibit 20-6):

$P(a_{O1}) = 1 - x_{O1} = 1 - 0.1387 = 0.8613$ (opposing westbound Lane 1)

$P(a_{O2}) = 1 - x_{O2} = 1 - 0.1458 = 0.8542$ (opposing westbound Lane 2)

$P(a_{CL1}) = 1 - x_{CL1} = 1 - 0.0427 = 0.9573$ (conflicting from left Lane 1)

$P(a_{CL2}) = 1 - x_{CL2} = 1 - 0.1102 = 0.8898$ (conflicting from left Lane 2)

$P(a_{CL3}) = 1 - x_{CL3} = 1 - 0.0782 = 0.9218$ (conflicting from left Lane 3)

$P(a_{CR1}) = 1 - x_{CR1} = 1 - 0.0676 = 0.9324$ (conflicting from right Lane 1)

$P(a_{CR2}) = 1 - x_{CR2} = 1 - 0.1458 = 0.8542$ (conflicting from right Lane 2)

$P(a_{CR3}) = 1 - x_{CR3} = 1 - 0.1031 = 0.8969$ (conflicting from right Lane 3)

Therefore,

$$\begin{aligned} P(1) &= P(a_{O1})P(a_{O2})P(a_{CL1})P(a_{CL2})P(a_{CL3})P(a_{CR1})P(a_{CR2})P(a_{CR3}) \\ &= (0.8613)(0.8542)(0.9573)(0.8898)(0.9218)(0.9324)(0.8542)(0.8969) \\ &= 0.4127 \end{aligned}$$

To complete the calculations for Step 7, the computations are completed for the remaining 511 possible combinations. The full computational results for the eastbound (Lane 1) can be seen in the "DepHdwyIter1" spreadsheet tab, Rows 3118–3629 (Columns C–K).

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 20-1 through Equation 20-5 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined (assuming the 512 cases presented in Exhibit 32-16):

$$P(C_1) = P(1)$$

$$P(C_2) = \sum_{i=2}^{8} P(i)$$

$$P(C_3) = \sum_{i=9}^{22} P(i)$$

$$P(C_4) = \sum_{i=23}^{169} P(i)$$

$$P(C_5) = \sum_{i=170}^{512} P(i)$$

Again for the example of eastbound (Lane 1), these computations are made by summing Rows 3118–3629 in the spreadsheet for each of the five cases (Columns R–V). The resulting computations are shown in Row 3630 (Columns R–V), where

$$P(C_1) = 0.4127$$

$$P(C_2) = 0.1482$$

$$P(C_3) = 0.2779$$

$$P(C_4) = 0.1450$$

$$P(C_5) = 0.0162$$

The probability adjustment factors are then computed with Equation 32-14 through Equation 32-18, where α equals 0.01 (or 0.00 if correlation among saturation headways is not taken into account).

For example, by using Equation 32-14:

$$AdjP(1) = 0.01[0.1482 + 2(0.2779) + 3(0.1450) + 4(0.0162)]/1 = 0.01204$$

The results of the remaining computations (Equation 32-15 to Equation 32-18) for eastbound Lane 1 are located in Row 3632 of the spreadsheet (Columns S–V).

Step 9: Compute Saturation Headways

The base saturation headways for each of the 512 combinations can be determined with Exhibit 20-14. They are adjusted by using the adjustment factors

calculated in Step 4 and added to the base saturation headways to determine saturation headways.

For the example of eastbound (Lane 1), these computations are shown in Rows 3118–3629 of the spreadsheet (Columns M–O).

Step 10: Compute Departure Headways

The departure headway of the approach is the sum of the products of the adjusted probabilities and the saturation headways. For the example of eastbound (Lane 1), these computations are made by summing the product of Columns O and Y for Rows 3118–3629 in the example spreadsheet.

Step 11: Check for Convergence

The calculated values of h_d are checked against the assumed initial values for h_d. After one iteration, each calculated headway differs from the initial value by more than 0.1 s. Therefore, the new calculated headway values are used as initial values in a second iteration. For this problem, five iterations were required for convergence.

	EB L1	EB L2	EB L3	WB L1	WB L2	WB L3	NB L1	NB L2	NB L3	SB L1	SB L2	SB L3
Total lane flow rate	56	216		156	164		76	164	116	48	124	88
hd, initial value, Iteration 1	3.2	3.2		3.2	3.2		3.2	3.2	3.2	3.2	3.2	3.2
x, initial, Iteration 1	0.0498	0.192		0.1387	0.1458		0.0676	0.1458	0.1031	0.0427	0.1102	0.0782
hd, computed value, Iteration 1	6.463	5.755		6.405	5.597		6.440	5.935	5.228	6.560	6.055	5.347
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 2	6.463	5.755		6.405	5.597		6.440	5.935	5.228	6.560	6.055	5.347
x, initial, Iteration 2	0.1005	0.3453		0.2776	0.255		0.136	0.2704	0.1685	0.0875	0.2086	0.1307
hd, computed value, Iteration 2	7.550	6.838		7.440	6.629		7.537	7.027	6.313	7.740	7.230	6.515
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 3	7.550	6.838		7.440	6.629		7.537	7.027	6.313	7.740	7.230	6.515
x, initial, Iteration 3	0.1174	0.4103		0.3224	0.302		0.1591	0.3201	0.2034	0.1032	0.249	0.1593
hd, computed value, Iteration 3	7.970	7.257		7.854	7.041		7.954	7.442	6.725	8.187	7.675	6.957
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 4	7.970	7.257		7.854	7.041		7.954	7.442	6.725	8.187	7.675	6.957
x, initial, Iteration 4	0.124	0.4354		0.3404	0.3208		0.1679	0.339	0.2167	0.1092	0.2643	0.17
hd, computed value, Iteration 4	8.130	7.416		8.010	7.196		8.114	7.601	6.884	8.359	7.845	7.126
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 5	8.130	7.416		8.010	7.196		8.114	7.601	6.884	8.359	7.845	7.126
x, initial, Iteration 5	0.1265	0.445		0.3471	0.3278		0.1713	0.3463	0.2218	0.1115	0.2702	0.1742
hd, computed value, Iteration 5	8.191	7.476		8.069	7.255		8.174	7.661	6.943	8.424	7.910	7.190
Convergence?	Y	Y		Y	Y		Y	Y	Y	Y	Y	Y

Step 12: Compute Capacity

As noted in the procedure, the capacity of each approach is computed by increasing the given flow rate on the subject lane (assuming the flows on the opposing and conflicting approaches are constant) until the degree of utilization for the subject lane reaches 1. This level of calculation requires running an iterative procedure many times, which is practical only for a spreadsheet or software implementation.

For this example, the capacity of eastbound Lane 1 can be found to be approximately 420 veh/h. This value is lower than the value that could be estimated by dividing the approach volume by the degree of utilization (56/0.1265 = 443 veh/h). The difference is due to the interaction effects among the approaches: increases in eastbound traffic volume increase the departure headways of the other approaches, which increases the departure headway of the subject approach.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time by using Equation 20-29. For the eastbound Lane 1 (using a value for *m* of 2.3 for Geometry Group 6), the calculation is as follows:

$$t_s = h_d - m = 8.19 - 2.3 = 5.89$$

Step 14: Compute Control Delay

The control delay for each approach is computed with Equation 20-30 as follows (eastbound Lane 1 illustrated):

$$d = 5.89 + 900(0.25)\left[(0.1274-1)+\sqrt{(0.1274-1)^2+\frac{(8.19)(0.1274)}{(450)(0.25)}}\right]+5 = 12.1\text{ s}$$

On the basis of Exhibit 20-2, eastbound Lane 1 is assigned LOS B.

Step 15: Compute Queue Length

The 95th percentile queue for each lane is computed with Equation 20-33 as follows for eastbound Lane 1:

$$Q_{95} \approx \frac{900(0.25)}{8.19}\left[(0.1274-1)+\sqrt{(0.1274-1)^2+\frac{(8.19)(0.1274)}{(150)(0.25)}}\right] = 0.4\text{ veh}$$

This queue length commonly would be rounded up to 1 vehicle.

Discussion

The overall results can be found in the "DelayLOS" spreadsheet tab. As indicated in the output, all movements at the intersection are operating well with small delays. The worst-performing movement is eastbound Lane 2, which is operating with a volume-to-capacity ratio of 0.45 and a control delay of 16.1 s/veh, which results in LOS C.

CHAPTER 33
ROUNDABOUTS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. SUPPLEMENTAL GUIDANCE

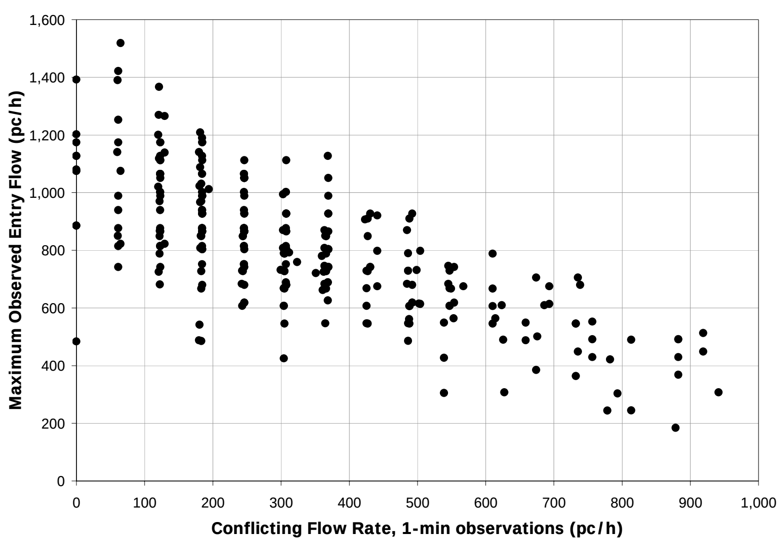
This section presents supplemental guidance on the methodology provided in Chapter 21, Roundabouts.

VARIABILITY AND UNCERTAINTY

The analyst should be aware of the large observed variation in driver behavior at roundabouts. Exhibit 33-1 shows observed combinations of entry flow and conflicting flow during 1-min periods of constant queuing and demonstrates the wide scatter of measured entry flows during capacity conditions. Part of this variation can be explained by the instability of 1-min measurements. The remainder of the variation is attributable primarily to variations in driver behavior, truck percentage, and exiting vehicles. Since there is no external control device regulating flow interactions at roundabouts, driver interactions govern the operation, and they are highly variable by nature.

Exhibit 33-1
Observed Combinations of Entry Flow and Conflicting Flow During 1-min Periods of Continuous Queuing at Single-Lane Roundabout Entries

LIVE GRAPH
Click here to view



Source: Data from *NCHRP Report 572* (*1*).

LANE-USE ASSIGNMENT

Lane-use assignment is best determined by measuring lane use in the field under the conditions being analyzed. In the absence of this information, default values or estimates can be used. This section provides background on the process by which an analyst can make an appropriate selection of a lane utilization factor.

There were few instances in the database used to develop the multilane model (*1*) in which a steady-state queue existed on all lanes of a multilane entry. Most commonly, for the two-lane entries, the outside lane had a sustained queue while the inside lane only had sporadic queuing. In general, several factors contribute to the assignment of traffic flow to each lane:

Turning movement patterns greatly influence lane assignments.

1. The assignment of turning movements to each lane (either as exclusive lanes or as shared lanes) directly influences the assignment of traffic

volumes to each lane. This is generally accomplished through the use of signs and pavement markings that designate the lane use for each lane. Multilane entries with no lane-use signing or pavement markings may be assumed to operate with a shared left-through lane in the left lane and a shared through-right lane in the right lane, although field observations should be made to confirm the lane-use pattern of an existing roundabout.

Dominant turning movements may create de facto lanes. A de facto lane is one designated for multiple movements but that may operate as an exclusive lane because of a dominant movement demand. A common example is a left-through lane with a left-turn flow rate that greatly exceeds the through flow rate.

2. Dominant turning movements may create de facto lane assignments where there is no advantage for drivers in using both lanes assigned to a given turning movement. For example, at an entry with left-through and through-right lanes and a dominant left-turn movement, there may be no advantage for through drivers in using the left lane. In addition, a lack of lane balance through the roundabout (e.g., two entry lanes but only one downstream circulating lane or one downstream exit lane) can create de facto lane-use assignments for a particular entry.

Downstream destinations may influence lane assignment.

3. Destinations downstream of a roundabout may influence the lane choice at the roundabout entry. A downstream destination such as a freeway on-ramp may increase use of the right entry lane, for example, even though both lanes could be used.

Poor geometric alignment of the entry may cause drivers to avoid the left lane.

4. The alignment of the lane relative to the circulatory roadway seems to influence the use of entry lanes where drivers could choose between lanes. Some roundabouts have been designed with rather perpendicular entries that have a natural alignment of the right entry lane into the left lane of the circulatory roadway. Under this design, the left entry lane is naturally aimed at the central island and is thus less comfortable and less desirable for drivers. This phenomenon, documented elsewhere (*2*, *3*) as *vehicle path overlap*, may result in poor use of the left entry lane. Similarly, poorly aligned multilane exits, where vehicles exiting in the inside lane cross the path of vehicles exiting in the outside lane, may influence lane use on upstream entries. In either case, the effect is most readily measured in the field at existing roundabouts and should be avoided in the design of new roundabouts.

Unfamiliar drivers may incorrectly select lanes for the intended movements.

5. Drivers may be uncertain about lane use when they use the roundabout, particularly at roundabouts without designated lane assignments approaching or circulating through the roundabout. This may contribute to the generally incorrect use of the right entry lane for left turns, for example, because of a perceived or real difficulty in exiting from the inside lane of the circulatory roadway. Proper signing and striping of lane use on the approach and through the roundabout may reduce this uncertainty, although it is likely to be present to some extent at multilane roundabouts.

Of these items, the first three factors are common to all intersections and are accounted for in the assignment of turning-movement patterns to individual lanes. Of the remaining two factors, both of which are unique to roundabouts, the fourth factor should be addressed through proper alignment of the entry relative to the circulatory roadway and thus may not need to be considered in

the analysis of new facilities. However, existing roundabouts may exhibit path overlap, resulting in poor lane utilization. It may be possible to reduce the fifth factor through proper design, particularly through lane-use arrows and striping. These collectively make accurate estimation of lane utilization difficult, but it may be measured at existing roundabouts.

Multilane roundabouts generally exhibit a bias to the right lane except where a double left-turn movement is present.

For entries with two through lanes, limited field data suggest that drivers generally have a bias for the right lane. For entries with two left-turn lanes (e.g., left-turn-only and shared left-through-right lanes), limited field data suggest that drivers have a bias for the left lane. Although no field observations have been documented for entries with two right-turn lanes, experience at other types of intersections with two right-turn lanes suggests that drivers have a bias for the right lane.

CAPACITY MODEL CALIBRATION

As discussed in Chapter 21, Roundabouts, the capacity model can be calibrated by using two parameters: the critical headway t_c and the follow-up headway t_f. One example application of this calibration procedure was performed for roundabouts in California (*4*).

Field-measured values for critical headway and follow-up headway were determined as follows:

- Critical headway:
  - Single-lane roundabouts: 4.8 s
  - Multilane roundabouts, left lane: 4.7 s
  - Multilane roundabouts, right lane: 4.4 s
- Follow-up headway:
  - Single-lane roundabouts: 2.5 s
  - Multilane roundabouts, left lane: 2.2 s
  - Multilane roundabouts, right lane: 2.2 s

By using these values and the expressions in Equation 21-21 through Equation 21-23, the capacity equation for single-lane roundabouts can be expressed as follows:

$$A = \frac{3{,}600}{t_f} = \frac{3{,}600}{2.5} = 1{,}440$$

$$B = \frac{t_c - (t_f/2)}{3{,}600} = \frac{4.8 - (2.5/2)}{3{,}600} = 1.0 \times 10^{-3}$$

$$c_{pce} = Ae^{(-Bv_c)} = 1{,}440e^{(-1.0\times10^{-3}v_c)}$$

Therefore, the model resulting from the use of California-specific data for critical headway and follow-up time has a higher intercept, and thus higher capacity, over its entire range than does the model based on the national study. These equations replace the equations in Step 5 of the Chapter 21 methodology.

2. REFERENCES

Many of these references can be found in the Technical Reference Library in Volume 4.

1. Rodegerdts, L., M. Blogg, E. Wemple, E. Myers, M. Kyte, M. P. Dixon, G. F. List, A. Flannery, R. Troutbeck, W. Brilon, N. Wu, B. N. Persaud, C. Lyon, D. L. Harkey, and D. Carter. *NCHRP Report 572: Roundabouts in the United States.* Transportation Research Board of the National Academies, Washington, D.C., 2007.
2. Robinson, B. W., L. Rodegerdts, W. Scarbrough, W. Kittelson, R. Troutbeck, W. Brilon, L. Bondzio, K. Courage, M. Kyte, J. Mason, A. Flannery, E. Myers, J. Bunker, and G. Jacquemart. *Roundabouts: An Informational Guide*. Report FHWA-RD-00-067. Federal Highway Administration, Washington, D.C., June 2000.
3. Rodegerdts, L., J. Bansen, C. Tiesler, J. Knudsen, E. Myers, M. Johnson, M. Moule, B. Persaud, C. Lyon, S. Hallmark, H. Isebrands, R. B. Crown, B. Guichet, and A. O'Brien. *NCHRP Report 672: Roundabouts: An Informational Guide*, 2nd ed. Transportation Research Board of the National Academies, Washington, D.C., 2010.
4. Tian, Z. Z., F. Xu, L. A. Rodegerdts, W. E. Scarbrough, B. L. Ray, W. E. Bishop, T. C. Ferrara, and S. Mam. *Roundabout Geometric Design Guidance*. Report No. F/CA/RI-2006/13. Division of Research and Innovation, California Department of Transportation, Sacramento, Calif., June 2007.

northbound right-turn movements of the adjacent intersection experience additional lost time of 2.10 and 3.07 s, respectively.

Exhibit 34-75
Example Problem 6: Lost Time due to Downstream Queues

Movement	Interchange EB EXT-TH	Interchange SB-L	Interchange WB EXT-TH	Interchange NB-L
V_R or V_A (veh/h)	191	805	216	822
N_R or N_A	1	2	1	2
G_R or G_A (s)	39	63	53	63
G_D (s)	97	97	111	111
C (s)	160	160	160	160
CG_{UD} or CG_{RD} (s)	53	34	53	53
Queue length (Q_A or Q_R) (ft)	0.0	0.0	0.0	0.0
Effective Green Adjustment	**Lost Time due to Downstream Queue — Interchange EB EXT-TH**	**SB-L**	**WB EXT-TH**	**NB-L**
G_R or G_A (s)	63	39	63	53
C (s)	160	160	160	160
D_{QA} or D_{QR} (ft)	500	500	500	500
CG_{UD} or CG_{RD} (s)	53	34	53	53
Additional lost time, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (s)	0.0	0.0	0.0	0.0
Total lost time, t'_L (s)	5.0	5.0	5.0	5.0
Effective green time, g' (s)	63	39	63	53

Movement	Adjacent Intersection EB-TH	Adjacent Intersection SB-L	Adjacent Intersection NB-R	Interchange WB INT-TH	Interchange SB-R
V_R or V_A (veh/h)	474	804	804	156	795
N_R or N_A	1	2	2	1	2
G_R or G_A (s)	48	59	59	39	111
G_D (s)	63	63	63	59	59
C (s)	160	160	160	160	160
CG_{UD} or CG_{RD} (s)	25.0	0.0	0.0	15	39
Queue length (Q_A or Q_R) (ft)	56.9	102.6	102.6	0.0	91.1
Effective Green Adjustment	**Lost Time due to Downstream Queue — Adjacent Intersection EB-TH**	**SB-L**	**NB-R**	**Interchange WB INT-TH**	**SB-R**
G_R or G_A (s)	59	24	24	119	39
C (s)	160	160	160	160	160
D_{QA} or D_{QR} (ft)	243	197	197	300	209
CG_{UD} or CG_{RD} (s)	25.0	29	0	15	39
Additional lost time, $L_{D\text{-}A}$ or $L_{D\text{-}R}$ (s)	0.00	2.10	3.07	0.0	0.0
Total lost time, t'_L (s)	5.00	7.10	8.07	5.0	5.0
Effective green time, g' (s)	59.0	21.9	20.9	119	39

Queue Storage and Control Delay

The queue storage ratio is estimated as the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 34-76 and Exhibit 34-77 present the calculations of the queue storage ratio for all approaches of the interchange, while Exhibit 34-78 gives the results of all approaches of the adjacent intersection. The *v*/*c* ratio for the respective movements is also provided in these exhibits.

Control delay for each movement is calculated according to Equation 18-47. Exhibit 34-79 through Exhibit 34-81 summarize the control delay estimates for all approaches of the interchange and adjacent signalized intersection.

Exhibit 34-76
Example Problem 6: Queue Storage Ratio for Interchange Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	888	99	897	961	219	820
s (veh/h/ln)	1,835	1,699	1,770	1,819	1,759	1,755
g (s)	63	29	97	63	43	111
g/C	0.39	0.18	0.61	0.39	0.27	0.69
I	1.00	0.75	0.75	1.00	0.68	0.68
c (veh/h/ln group)	1,448	308	2,146	1,448	473	2,435
$X = v/c$	0.61	0.32	0.42	0.66	0.46	0.34
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.333	1.000	1.000	1.333
P	0.39	0.18	0.81	0.39	0.27	0.92
r (s)	97.00	131.00	63.00	97.00	117.00	49.00
t_f (s)	0.01	0.00	0.00	0.01	0.00	0.00
q (veh/s)	0.25	0.03	0.25	0.27	0.06	0.23
q_g (veh/s)	0.25	0.03	0.33	0.27	0.06	0.30
q_r (veh/s)	0.25	0.03	0.12	0.27	0.06	0.06
Q_1 (veh)	13.8	3.5	8.5	13.0	7.3	1.1
Q_2 (veh)	0.8	0.2	0.1	1.1	0.3	0.1
T	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00
t_A	0	0	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	14.6	3.7	8.6	14.1	7.6	1.2
L_h (ft)	25	25	25	25	25	25
L_a (ft)	600	200	500	600	200	500
R_Q	0.61	0.46	0.43	0.59	0.95	0.06

Exhibit 34-77
Example Problem 6: Queue Storage Ratio for Interchange Northbound and Southbound Movements

	Northbound Movements		Southbound Movements	
	Left	Right	Left	Right
Q_{bL} (ft)	0.0	0.0	0.0	0.0
v (veh/h/ln group)	216	210	191	161
s (veh/h/ln)	1,682	1,651	1,669	1,634
g (s)	53	53	39	39
g/C	0.33	0.33	0.24	0.24
I	1.00	1.00	1.00	1.00
c (veh/h/ln group)	557	547	407	398
$X = v/c$	0.39	0.38	0.47	0.40
r_a (ft/s^2)	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4
S_s (mi/h)	5	5	5	5
S_{pl} (mi/h)	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.000	1.000
P	0.33	0.33	0.24	0.24
r (s)	107.00	107.00	121.00	121.00
t_f (s)	0.00	0.00	0.00	0.00
q (veh/s)	0.06	0.06	0.05	0.04
q_g (veh/s)	0.06	0.06	0.05	0.04
q_r (veh/s)	0.06	0.06	0.05	0.04
Q_1 (veh)	6.6	6.4	6.5	5.4
Q_2 (veh)	0.3	0.3	0.4	0.3
T	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00
t_A	0	0	0	0
Q_e (veh)	0.00	0.00	0.00	0.00
Q_b (veh)	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0
Q (veh)	6.9	6.7	7.0	5.7
L_h (ft)	25	25	25	25
L_a (ft)	400	400	400	400
R_Q	0.43	0.42	0.43	0.36

Exhibit 34-78
Example Problem 6: Queue Storage Ratio for Adjacent Intersection Movements

	Eastbound		Westbound		Northbound			Southbound	
	Through & Right	Left	Through & Right	Left	Through	Right	Left	Through & Right	Left
Q_{bL} (ft)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
v (veh/h/ln group)	866	227	577	309	206	186	108	542	289
s (veh/h/ln)	1,679	1,680	1,650	1,722	1,765	1,580	1,568	1,717	1,654
g (s)	59.0	33	59	33	24.0	20.9	24.0	24	21.9
g/C	0.37	0.21	0.37	0.21	0.15	0.13	0.15	0.15	0.14
I	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
c (veh/h/ln group)	1,288	346	1,218	355	265	237	235	515	248
$X = v/c$	0.67	0.65	0.47	0.46	0.78	0.90	0.46	1.05	1.28
r_a (ft/s^2)	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
r_d (ft/s^2)	4	4	4	4	4	4	4	4	4
S_s (mi/h)	5	5	5	5	5	5	5	5	5
S_{pl} (mi/h)	40	40	40	40	40	40	40	40	40
S_a (mi/h)	39.96	39.96	39.96	39.96	39.96	39.96	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04	12.04	12.04	12.04	12.04	12.04	12.04
Rp	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
P	0.37	0.21	0.37	0.21	0.15	0.13	0.15	0.15	0.14
r (s)	101.00	127.00	101.00	127.00	136.00	139.07	136.00	136.00	138.10
t_r (s)	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.01	0.01
q (veh/s)	0.24	0.06	0.16	0.04	0.06	0.05	0.03	0.08	0.08
q_g (veh/s)	0.24	0.06	0.16	0.04	0.06	0.05	0.03	0.08	0.08
q_r (veh/s)	0.24	0.06	0.16	0.04	0.06	0.05	0.03	0.08	0.08
Q_1 (veh)	14.3	8.4	8.7	5.5	8.0	7.4	4.0	10.4	9.2
Q_2 (veh)	1.1	0.9	0.5	0.4	1.5	2.3	0.4	5.0	9.7
T	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.39	15.6
t_A	0	0	0	0	0	0	0	0.25	0.25
Q_e (veh)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.39	15.6
Q_b (veh)	0	0	0	0	0	0	0	0	0
Q_3 (veh)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Q (veh)	15.4	9.3	9.1	5.9	9.5	9.8	4.4	15.5	18.8
L_h (ft)	25	25	25	25	25	25	25	25	25
L_a (ft)	800	200	300	200	800	800	200	800	200
R_Q	0.48	1.16	0.76	0.73	0.30	0.30	0.55	0.48	2.36

Exhibit 34-79
Example Problem 6: Control Delay for Interchange Eastbound and Westbound Movements

	Eastbound Movements			Westbound Movements		
	EXT-TH&R	INT-L	INT-TH	EXT-TH&R	INT-L	INT-TH
g (s)	-	29	97	-	43	111
g' (s)	63	-	-	63	-	-
g/C or g'/C	0.39	0.18	0.61	0.39	0.27	0.69
c (veh/h)	1,448	308	2,146	1,448	473	2,435
$X = v/c$	0.61	0.32	0.42	0.68	0.46	0.34
d_1 (s/veh)	38.8	56.9	16.6	30.6	48.8	2.0
k	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	3.9	2.1	0.5	5.4	2.2	0.3
d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0
PF	1.000	1.000	0.560	1.000	1.000	0.283
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0
t	0	0	0	0	0	0
d (s/veh)	42.6	59.0	17.1	36.0	51.0	2.2

Exhibit 34-80
Example Problem 6: Control Delay for Interchange Northbound and Southbound Movements

	Northbound Movements Left	Northbound Movements Right	Southbound Movements Left	Southbound Movements Right
g (s)	-	53	-	39
g' (s)	53	-	39	-
g/C or g'/C	0.33	0.33	0.24	0.24
c (veh/h)	557	547	407	398
$X = v/c$	0.39	0.38	0.47	0.40
d_1 (s/veh)	41.1	41.0	51.7	50.7
k	0.5	0.5	0.5	0.5
d_2 (s/veh)	2.0	2.0	3.8	3.0
d_3 (s/veh)	0.0	0.0	0.0	0.0
PF	1	1	1	1
k_{min}	0.04	0.04	0.04	0.04
u	0	0	0	0
t	0	0	0	0
d (s/veh)	43.1	43.0	55.5	53.8

Exhibit 34-81
Example Problem 6: Control Delay for Adjacent Intersection Movements

	Eastbound Through & Right	Eastbound Left	Westbound Through & Right	Westbound Left	Northbound Through	Northbound Right	Northbound Left	Southbound Through & Right	Southbound Left
g (s)	-	33.0	59.0	33.0	24.0	-	24.0	24.0	-
g' (s)	59.0	-	-	-	-	20.9	-	-	21.9
g/C or g'/C	0.37	0.21	0.37	0.21	0.15	0.13	0.15	0.15	0.14
c (veh/h)	1,288	346	1,218	355	265	237	235	258	248
$X = v/c$	0.67	0.65	0.47	0.87	0.78	0.78	0.46	1.05	1.28
d_1 (s/veh)	42.5	58.3	38.7	55.6	65.4	68.5	62.1	68.0	69.0
k	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
d_2 (s/veh)	6.0	9.3	2.7	4.4	20.0	40.7	6.4	70.6	153.6
d_3 (s/veh)	0	0	0	0	0	0	0	0	0
PF	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
k_{min}	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
u	0	0	0	0	0	0	0	0	0
t	0	0	0	0	0	0	0	0	0
d (s/veh)	48.5	67.6	41.4	60.0	85.4	109.1	68.4	138.6	226.6

Results

Delay for each O-D is estimated as the sum of the movement delays for each movement utilized by the O-D, as indicated in Equation 22-1. The *v/c* and queue storage ratios are checked next. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement is F. The final delay calculations and resulting LOS for each O-D and each lane group are presented in Exhibit 34-82 and Exhibit 34-83. As shown, the *v/c* ratio and R_Q for all O-Ds are all below 1, and therefore the LOS for all O-Ds is determined by using Exhibit 22-11. The LOS for each lane group at the adjacent intersection is assigned on the basis of Chapter 18, Signalized Intersections.

Exhibit 34-82
Example Problem 6: Interchange O-D Movement LOS

O-D Movement	Control Delay (s)	*v/c* > 1?	R_Q > 1?	LOS
A	45.3	No	No	C
B	43.0	No	No	C
C	53.8	No	No	C
D	72.6	No	No	D
E	98.1	No	No	E
F	39.1	No	No	C
G	36.0	No	No	C
H	87.0	No	No	E
I	56.2	No	No	D
J	38.2	No	No	C
K	-	-	-	-
L	-	-	-	-
M	94.1	No	No	E
N	114.5	No	No	E

Exhibit 34-83
Example Problem 6: Adjacent Intersection Movement LOS

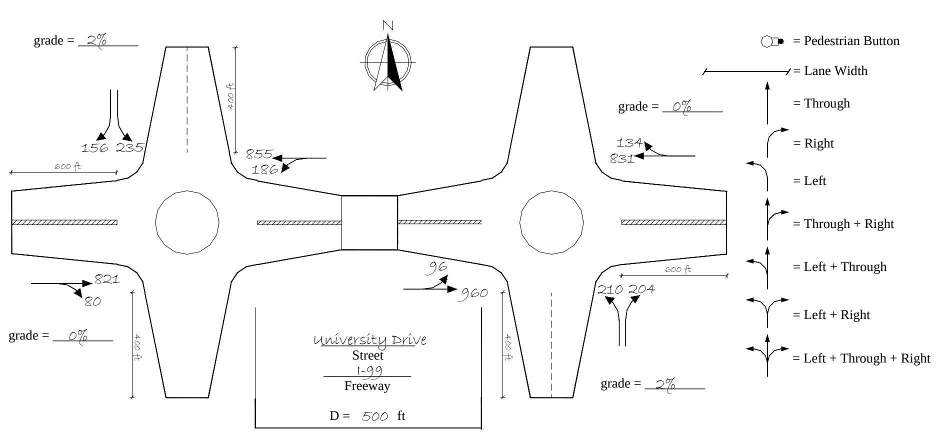
Approach	Lane Group	Control Delay (s)	LOS
EB	Through and right	48.5	C
	Left	67.6	D
WB	Through and right	41.4	C
	Left	60.0	D
NB	Through	85.4	E
	Right	109.1	E
	Left	68.4	D
SB	Through and right	138.6	F
	Left	226.6	F

EXAMPLE PROBLEM 7: DIAMOND INTERCHANGE WITH ROUNDABOUTS

The Interchange

The interchange of I-99 (NB/SB) and University Drive (EB/WB) is a diamond interchange featuring roundabouts. The traffic conditions of the interchange are provided in Exhibit 34-84.

Exhibit 34-84
Example Problem 7: Intersection Plan View



The Question

What are the control delay and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange. This interchange has 3% heavy vehicles, and the PHF of the interchange is 0.97. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). The results of the O-D calculations and the resulting PHF-adjusted values are presented in Exhibit 34-85.

Exhibit 34-85
Example Problem 7: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)	HV-Adjusted Demand (pc/h)
A	179	185	191
B	169	174	179
C	122	126	130
D	228	235	242
E	93	96	99
F	78	80	82
G	94	97	100
H	119	123	127
I	509	525	541
J	529	545	561
K	0	0	0
L	0	0	0
M	0	0	0
N	0	0	0

Calculation of Approach Capacity and Control Delay

To estimate the delay of each approach to the roundabout, the procedures outlined in Chapter 21, Roundabouts, are used to estimate the entering and conflicting flow rates and the resulting capacity of each approach.

Exhibit 22-26 and Exhibit 22-27 are used to determine the entering and conflicting flow rates for each approach of the interchange. For example, the northbound ramp movement (Number 13 in Exhibit 22-26) consists of O-D Movements A, B, K, and M at a diamond interchange (Exhibit 22-27). The conflicting flow (Number 12) consists of O-D Movements D, E, I, and N. Exhibit 34-86 shows the entering and conflicting flow for each approach, along with the corresponding capacity and delay.

Exhibit 34-86
Example Problem 7: Approach Capacity and Delay Calculations

Approach	Entering Flow (pc/h)	Conflicting Flow (pc/h)	Capacity (pc/h)	Control Delay (s/veh)
EB EXT	722	369	782	34.5
EB INT	882	0	1,130	13.4
WB EXT	788	289	846	33.8
WB INT	879	0	1,130	13.3
NB RAMP	370	882	468	30.9
SB RAMP	372	879	469	31.1

O-D Movement Control Delay and LOS

Delay for each O-D is estimated as the sum of the approach delays for each approach utilized by the O-D. For example, O-D Movement A will utilize the northbound ramp approach and the westbound internal through approach. The control delays for these approaches are then summed to estimate the control delay for O-D Movement A. LOS for each O-D is assigned on the basis of Exhibit 22-13. The resulting control delay and LOS for all O-D movements are shown in Exhibit 34-87.

Exhibit 34-87
Example Problem 7: Control Delay and LOS for Each O-D Movement

O-D	Control Delay (s/veh)	LOS
A	44.2	D
B	30.9	C
C	31.1	C
D	44.5	D
E	47.9	D
F	34.5	C
G	33.8	C
H	47.1	D
I	47.9	D
J	47.1	D
K	30.9	C
L	31.1	C
M	44.2	D
N	44.5	D

EXAMPLE PROBLEM 8: ALTERNATIVE ANALYSIS TOOL

This example presents a simulation analysis of the basic diamond interchange configuration originally described in Example Problem 1 of Chapter 22. A few changes were made to introduce elements that are beyond the stated limitations of the procedures presented in that chapter. The use of a typical simulation tool to address the limitations is described in this section.

The need to determine performance measures from an analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, the example presented in this chapter has applied existing versions of alternative tools and therefore does not reflect the trajectory-based measures described in Chapter 7.

Operational Characteristics

A two-way STOP-controlled (TWSC) intersection was introduced 600 ft west of the first signalized intersection of the interchange. Ramp metering signals were installed on both of the freeway entrance ramps. Right-turn storage bays

were introduced on all approaches to the interchange that accommodated right turns. The demand volumes were modified to introduce conditions that varied from undersaturated to heavily oversaturated. The signal timing plan was modified to accommodate the distribution of volumes.

Exhibit 34-88 shows the interchange configuration and demand volumes. The demand volumes are referenced to the total directional arterial demand *d*, which varies from 600 to 1,800 veh/h. The turning movement volumes entering and leaving the arterial have been balanced for continuity of traffic flow. The turning movements entering and leaving the freeway were set at 25% of the total approach volumes and were adjusted proportionally to match the arterial demand volumes. The cross-street entry demand from the TWSC intersection was held constant at 100 veh/h in each direction, with 50% assigned to the left and right turns. No through vehicles were assigned from the cross street at this intersection.

Exhibit 34-88
Example Problem 8: Interchange Configuration and Demand Volumes

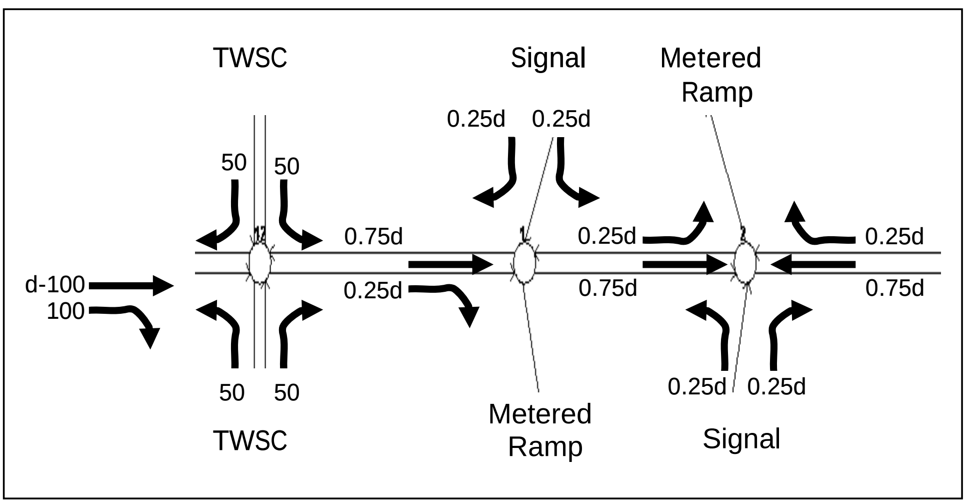


Exhibit 34-89 shows the signal timing plan for both intersections of the diamond interchange. A simple three-phase operation at each intersection is depicted in this table. No attempt has been made to optimize the phasing or timing since the main purpose of this example is to demonstrate self-aggravating phenomena that are not recognized by the Chapter 22 procedures. The ramp metering signals installed on each of the entrance ramps were set to release a single vehicle at 10-s intervals, giving a capacity of 360 veh/h for each ramp.

Exhibit 34-89
Example Problem 8: Signal Timing Plan

Movement	Green	Yellow	All Red
Entry through/left	20	4	1
Entry and exit through/right	45	4	1
Ramp	20	4	1
Cycle length		100 s	

Summary of Simulation Runs

The operation of this interchange was simulated by using demand volumes *d* ranging from 600 veh/h (very undersaturated) to 1,800 veh/h (very oversaturated). The volume increment was 200 veh/h. Thirty simulation runs

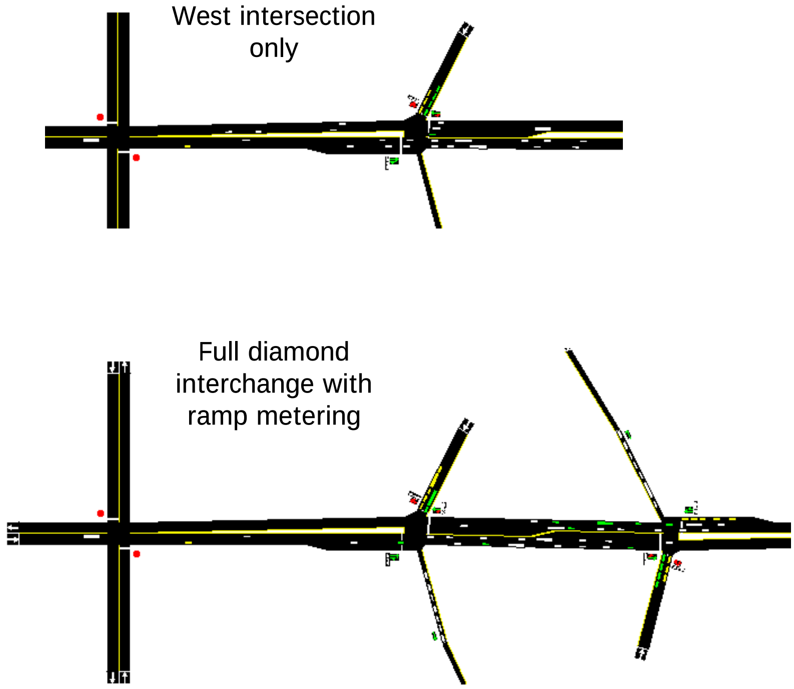
were made for each condition to accommodate the stochastic variation inherent in simulation runs.

Two configurations were examined for each of the demand levels:

1. A single intersection at the west end of the diamond interchange, and
2. The full diamond interchange with ramp metering.

Both of these configurations are illustrated in Exhibit 34-90.

Exhibit 34-90
Example Problem 8: Physical Configurations Examined



The west intersection was examined separately to demonstrate the differences between a signalized intersection operating independently and one that operates as part of a diamond interchange with mutual interactions between the intersections at each end.

Diamond Interchange Operation

Exhibit 34-91 illustrates the self-aggravating effects from interactions among the two signals that make up the interchange and the ramp metering. Backup and congestion are observed at high demands on all approaches. The left-turn bays on the internal interchange segments spill over to block through traffic. Backup from the ramp metering signals causes additional impediment to traffic trying to leave the interchange.

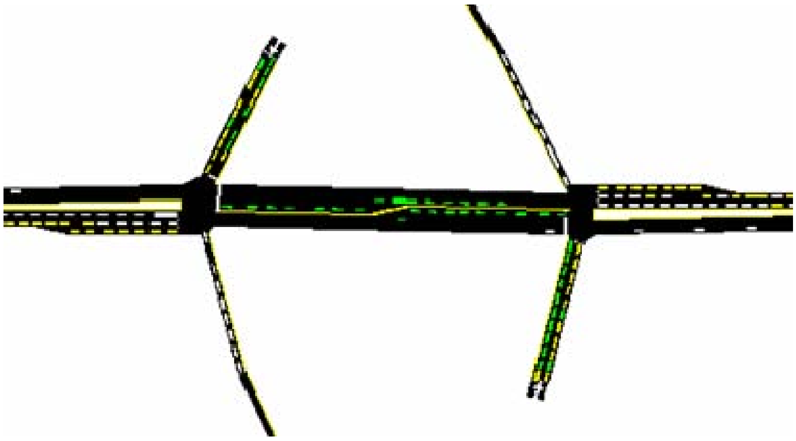


Exhibit 34-91
Example Problem 8: Congested Approaches to Diamond Interchange

Excessive delays will be associated with the oversaturated operation. However, for purposes of this example, the reduction in capacity is of more interest because capacity reductions due to self-aggravating phenomena are not fully recognized by the Chapter 22 methodology. Proper assessment of delay with heavy oversaturation would require a more complex procedure involving multiperiod analysis with possible consideration of route diversion due to the excessive congestion. Therefore, this example will be limited to examining the capacity reduction that results from interaction between the elements within this system. The extent of the capacity reduction will be estimated by the relationship between demand (input) and discharge (output) on the various segments.

Exhibit 34-92 shows the westbound arterial discharge from the diamond interchange (through plus left turns) as a function of the arterial demand *d*. Note that the discharge tracks the demand at low volumes, indicating that all of the arrivals were accommodated. As the demand increases, the discharge levels off at a point that indicates the capacity of the approach. When the approach is a part of an isolated intersection, the capacity approaches about 1,600 veh/h. A much lower capacity (about 850 veh/h) is attainable in the case of the diamond interchange with ramp metering. A number of self-aggravating phenomena reduce the capacity. Some westbound vehicles are unable to enter the east intersection because of backup from internal westbound left-turn bay spillover. Other westbound vehicles are unable to exit the interchange because of backup from the ramp metering signal and because of blockage of the intersection by left-turning exit ramp vehicles. The net result is a substantial reduction in capacity that would not be evident from application of the Chapter 22 methodology.

Exhibit 34-92
Example Problem 8:
Discharge from the Diamond
Interchange Under the Full
Range of Arterial Demand

LIVE GRAPH
Click here to view

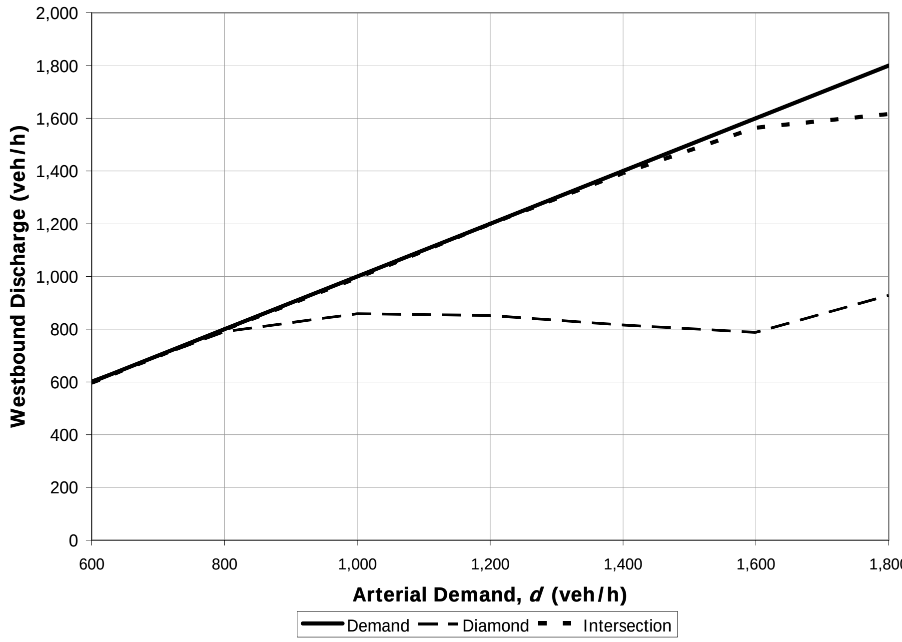
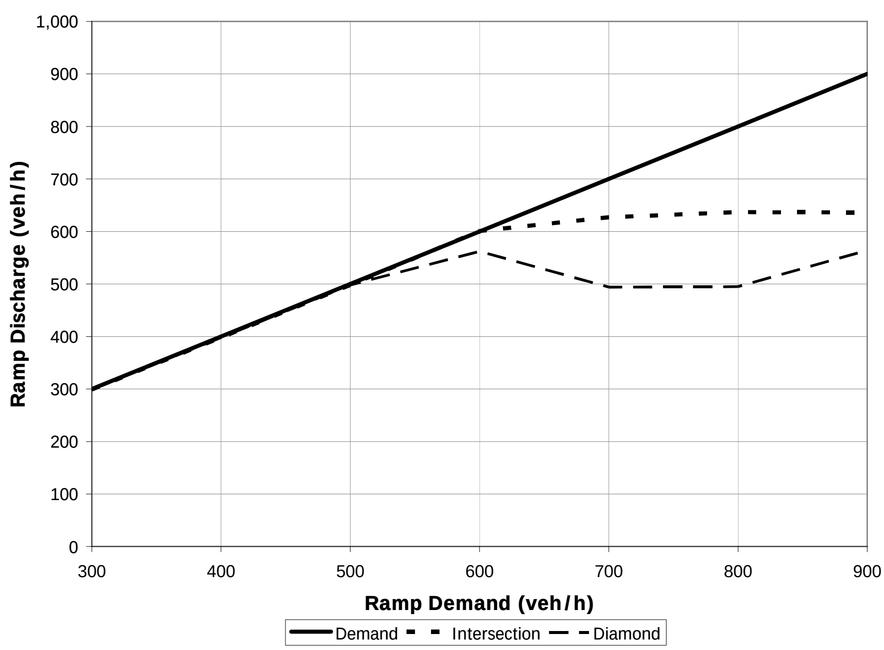


Exhibit 34-93 shows the effect of the demand volume on the southbound exit ramp discharge at the west signal of the diamond interchange. With an isolated signal, the discharge levels off at the approach capacity. As shown, the capacity is reduced slightly when the signal is part of a diamond interchange. The reduction was not as apparent as it was for the arterial movements because the blockage effects are not as significant. Some left turns were unable to enter the intersection because of backup from the east signal. The right turns from the ramp were not subject to any blockage effects.

Exhibit 34-93
Example Problem 8:
Discharge from the
Southbound Exit Ramp
Under the Full Range of
Ramp Demand

LIVE GRAPH
Click here to view



TWSC Intersection Operation

The TWSC analysis procedures prescribed in Chapter 19 recognize the effects of adjacent signalized intersections to some extent, but they do not address cases when an approach is blocked throughout part of a cycle by stationary queues that prevent vehicles from entering on the minor street. This situation is depicted in Exhibit 34-94, in which a stationary queue of eastbound vehicles backed up from the west intersection of the diamond interchange has blocked the entry to the intersection for three of the four minor street movements.

Exhibit 34-94
Example Problem 8: Congested Approaches to the TWSC Intersection

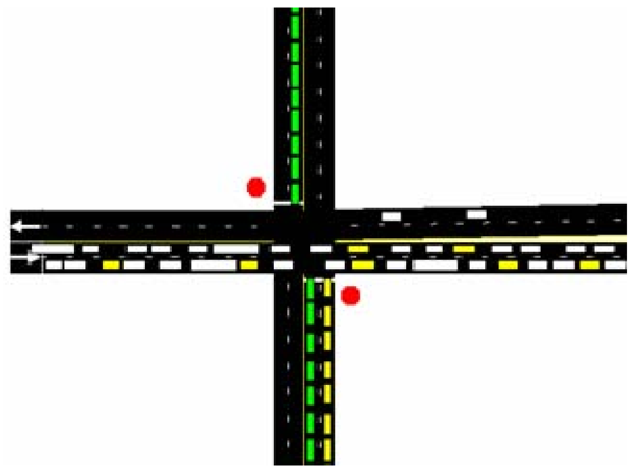
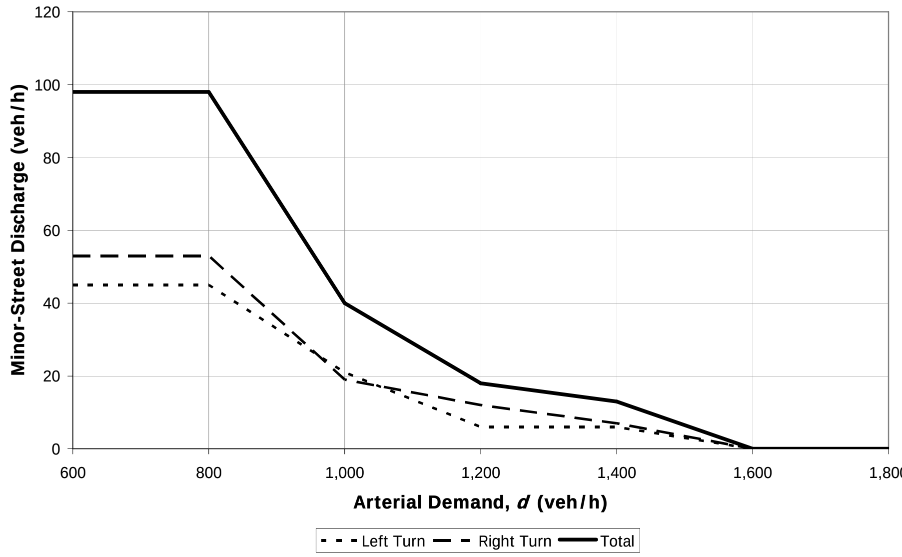


Exhibit 34-95 shows the minor-street entry as a function of the arterial demand. Unlike the other movements in this example, the minor-street demand was kept constant throughout the entire range of arterial demand. It is a well-established principle of TWSC analysis that the entry capacity for the minor-street movements diminishes with increasing major-street volumes. That phenomenon is depicted clearly for northbound traffic in Exhibit 34-94. It is evident here that the capacity begins to drop below the demand at about 800 veh/h in each direction on the arterial. The southbound situation, on the other hand, presents some surprising results. The southbound left turn is impeded by the queue of westbound vehicles backed up from the interchange, as expected. The southbound right turn, assisted by gaps created by the interchange signal, experiences an increase in capacity that produces entry volumes that exceed the original demand. The animated graphics display indicates that some of the southbound left-turn vehicles were unable to maneuver into the proper lane. The driver behavior model of the simulation tool reassigned these vehicles to right turns because of excessive waiting times. This effect provides a clear example of the difference between simulation modeling and the analytical approach presented throughout the HCM.

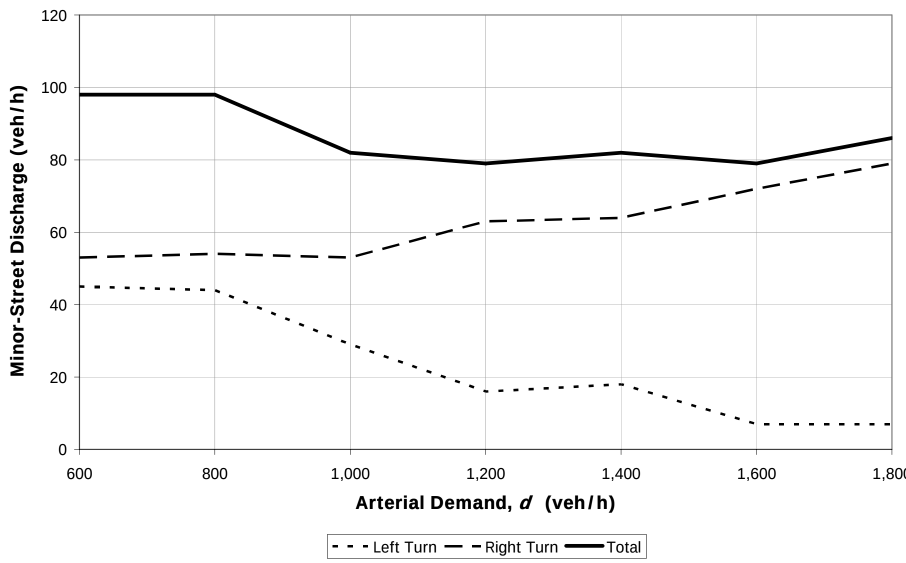
Exhibit 34-95
Example Problem 8: Effect of Arterial Demand on Minor-Street Discharge at the TWSC Intersection

LIVE GRAPH
Click here to view

LIVE GRAPH
Click here to view



(a) Northbound



(b) Southbound

CHAPTER 35
ACTIVE TRAFFIC MANAGEMENT

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

This first-generation chapter is intended to lead to more specific guidance in future HCM updates.

Active traffic management (ATM) is a comprehensive approach to optimizing the operational performance of the roadway system through monitoring and control of systems operations. ATM incorporates both demand and supply management strategies. Management of both demand and supply greatly enhances the ability of the transportation agency to achieve its system performance goals.

ATM can range from the simple to the complex. It may be relatively static, with routine monitoring of system performance and periodic changes to system controls in response to those measurements, or it may be highly dynamic, using sophisticated technology to update system controls continuously and automatically in response to real-time information on system conditions.

This chapter focuses on the following major ATM strategies:

- Roadway metering,
- Congestion pricing,
- Traveler information systems,
- Managed lanes,
- Traffic signal control, and
- Speed harmonization.

ATM strategies, however, are evolving as quickly as the technologies they employ. The above list is illustrative, not definitive, of ATM. New ATM strategies and variations are created with every advance in detection, communications, and control technology.

ATM strategies may be significant components of incident management plans, work zone management plans, and employer-based demand management programs.

PURPOSE

This chapter is ultimately intended to provide recommended methodologies and measures of effectiveness (MOEs) for evaluating the impacts of ATM strategies on highway and street system demand, capacity, and performance. However, at this time available information on the performance of ATM strategies has not matured sufficiently to enable the development and presentation of specific analysis methodologies. Consequently, this chapter limits itself to describing ATM strategies; discussing the mechanisms by which they affect demand, capacity, and performance; and offering general guidance on possible evaluation methods for ATM techniques. Later generations of this chapter will provide more specific guidance on the evaluation of ATM strategies.

ORGANIZATION

This chapter is organized as follows:

Introduction—Describes the chapter's scope, purpose, limitations, and organization.

ATM Strategies—Provides an overview of ATM strategies.

Meta–Measures of Effectiveness—Presents recommended meta–measures of effectiveness (meta-MOEs) that build on traditional *Highway Capacity Manual* (HCM) measures for assessing the effectiveness of ATM strategies.

Effectiveness—Serves as a stand-in for future sections on methodology and applications. It gives a general description of the mechanisms by which ATM strategies can affect demand, capacity, and performance; summarizes available evidence on the effects; and suggests possible analysis tools.

SCOPE AND LIMITATIONS

This chapter presents introductory information on ATM strategies and their effect on demand, capacity, and system performance.

Because research on ATM is still in its infancy, no specific methodologies are presented for evaluating the effects of ATM strategies. As of this writing a good deal of research on ATM strategies, methodologies, and MOEs is under way at the federal and state levels; the analyst is advised to consult the original research to better understand the basis and limitations of the tentative results cited in this chapter.

2. ACTIVE TRAFFIC MANAGEMENT STRATEGIES

OVERVIEW

This section provides brief overviews of typical ATM strategies for managing demand, capacity, and performance for the highway and street system. The strategies described here are intended to be illustrative rather than definitive. ATM strategies are constantly evolving with each advance in technology.

ROADWAY METERING

Roadway metering treatments store surges in demand at various points in the transportation network. Typical examples of roadway metering include freeway on-ramp metering, freeway-to-freeway ramp metering, freeway mainline metering, peak period freeway ramp closures, and arterial signal metering. Exhibit 35-1 illustrates a freeway ramp-metering application.

Exhibit 35-1
Freeway Ramp Metering, SR-94, Lemon Grove, California

Source: Federal Highway Administration, *Ramp Management and Control: A Primer* (*1*).

Roadway metering may be highly dynamic or comparatively static. A comparatively static roadway metering system would establish preset metering rates on the basis of historical demand data, periodically monitor system performance, and adjust the rates to obtain satisfactory facility performance. A highly dynamic system may monitor system performance and automatically adjust metering rates on a real-time basis by using a predetermined algorithm in response to changes in observed facility conditions.

Preferential treatment of high-occupancy vehicles (HOVs) may be part of a roadway metering strategy.

Roadway metering may be applied on freeways or arterials. An upstream signal may be used on arterials to control the number of vehicles reaching downstream signals. Surges in demand are temporarily stored at the upstream signal and released when the downstream signals can better serve the vehicles.

CONGESTION PRICING

The objective of congestion pricing is to preserve reliable operating speeds on the tolled facility.

Congestion or value pricing is the practice of charging tolls for use of all or part of a facility or a central area according to the severity of congestion. The objective of congestion pricing is to preserve reliable operating speeds on the tolled facility with a tolling system that encourages drivers to switch to other times of the day, other modes, or other facilities when demand starts to approach facility capacity. Exhibit 35-2 shows an example of congestion pricing in Minnesota.

Exhibit 35-2
Minnesota Dynamic Pricing for HOT Lanes

Source: Federal Highway Administration, *Technologies That Complement Congestion Pricing* (*2*) (courtesy of Minnesota Department of Transportation).

The tolls may vary by distance traveled, vehicle class, and estimated time savings. Tolls may be collected by either electronic or manual means, or both.

Congestion pricing may employ different degrees of responsiveness and automation. Some implementations may use a preset schedule in which the toll varies by the same amount for preset times during the day and week. The implementation may be monitored on a regular schedule and the pricing adjusted to achieve or maintain desired facility performance. An advanced implementation of congestion pricing may monitor facility performance more frequently and use automatic or semiautomatic dynamic pricing, varying the toll by using a predetermined algorithm according to the observed performance of the facility.

High-occupancy toll (HOT) lanes (sometimes called express lanes) are tolled lanes adjacent to general-purpose lanes. Motorists pay tolls to enter the HOT

lanes to avoid congested nontoll lanes. HOVs may be allowed to enter the lanes for free or at a reduced toll rate.

Central area pricing is an areawide implementation of congestion pricing.

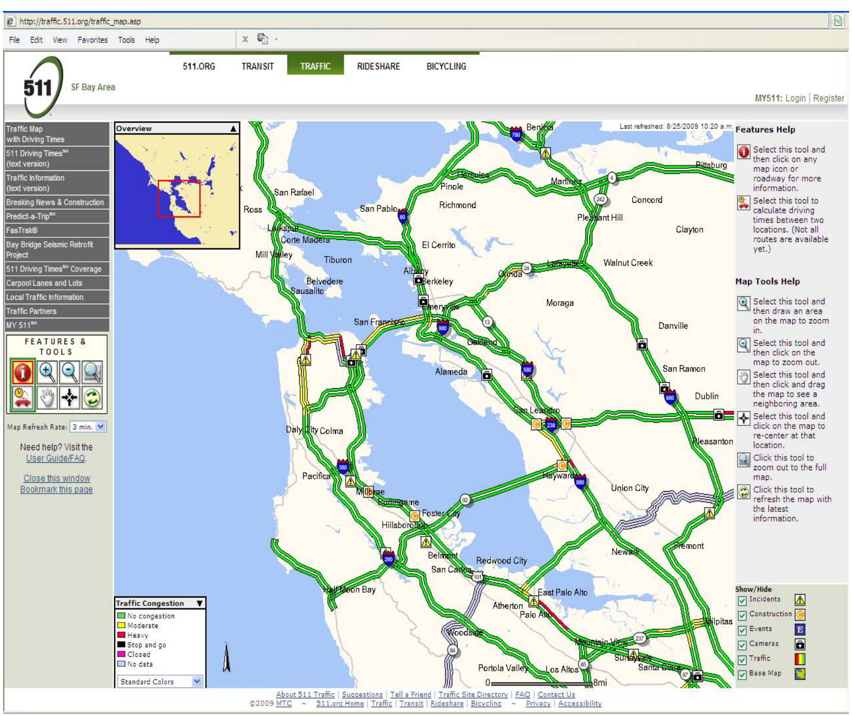
Central area pricing is an areawide implementation of congestion pricing that imposes tolls for vehicles both entering and traveling within a central area street network during certain hours of certain days. The fee varies by time of day, by day of week, or according to real-time measurements of congestion within the central area. The toll may be reduced or waived for certain vehicle types, such as HOVs, or for residents of the zone.

TRAVELER INFORMATION SYSTEMS

Traveler information is an integration of technologies that allow the general public to access real-time or near real-time data on traffic factors such as incident conditions, travel time, and speed. Traveler information systems can be divided into three types (pretrip, in-vehicle, and roadside) according to when the information is made available and how it is delivered to the driver.

Pretrip information is obtained from various sources and is transmitted to motorists before the start of their trip through various means. Exhibit 35-3 illustrates internet transmission of travel information.

Exhibit 35-3
San Francisco Bay Area Traffic Map



Source: Metropolitan Transportation Commission, copyright 2009. http://traffic.511.org.

In-vehicle information may involve route guidance or transmission of incident and travel time conditions to the vehicle while en route. Route guidance involves global positioning system–based real-time data acquisition to calculate the most efficient routes for drivers. This technology allows individual drivers to receive optimal route guidance and provides a method for the transportation network operator to make direct and reliable control decisions to stabilize network flow.

Roadside messages consist of dynamic message signs (also called changeable or variable message signs) and highway advisory radio (also called traveler advisory radio) that display or transmit information on road conditions for travelers while they are en route.

MANAGED LANES

The term "managed lanes" has historically referred to a broad range of ATM strategies related to the control of specific lane operations on a facility. That definition is retained here, but to avoid overlap, only those managed lane strategies not covered elsewhere in this chapter are described in the Managed Lanes section.

Managed lane strategies include reversible lanes, HOV lanes, HOT lanes, truck lanes, speed harmonization, temporary closures for incidents or maintenance, and temporary use of shoulders during peak periods (see Exhibit 35-4). HOT lanes are described above under congestion pricing; speed harmonization is described in a later section.

HOV lanes assign limited vehicle capacity to vehicles that carry the most people on the facility or that in some other way meet societal objectives for reducing the environmental impacts of vehicular travel (e.g., motorcycles or two-seater, electric, or hybrid vehicles). HOV lanes may operate 24 hours a day, 7 days a week, or may be limited to peak periods when demand is greatest. The minimum vehicle occupancy requirement for HOV lanes may be adjusted in response to operating conditions in the HOV lanes to preserve uncongested operation.

Exhibit 35-4
HOV Lane

Source: Federal Highway Administration, *Managed Lanes: A Primer* (*3*).

Reversible lanes provide additional capacity for directional peak flows depending on the time of the day. Reversible lanes on freeways may be located in the center of a freeway with gate control on both ends. On interrupted-flow facilities, reversible lanes may be implemented with the help of lane-use control signals and signs that open and close lanes by direction.

The temporary use of shoulders during peak periods by all or a subset of vehicle types can provide additional capacity in a bottleneck section and improve overall facility performance. Temporary shoulder use by transit vehicles in

queuing locations can reduce delays for those vehicles by enabling them to reach their exit without having to wait in the mainline queue.

SPEED HARMONIZATION

The objective of speed harmonization is to improve safety and facility operations by reducing the shock waves that typically occur when traffic abruptly slows upstream of a bottleneck or for an incident. The reduction of shock waves decreases the probability of secondary incidents and the loss of capacity associated with incident-related and recurring traffic congestion.

Changeable speed limit or speed advisory signs are typically used to implement speed harmonization. The speed restrictions may apply uniformly across all lanes or may vary by lane. Although not strictly a speed harmonization technique, the same lane signs may be used to close individual lanes upstream of an incident until the incident is cleared.

The variable speed limit may be advisory or regulatory. Advisory speeds indicate a recommended speed that drivers may exceed if they believe it is safe under prevailing conditions. Regulatory speed limits may not be exceeded under any conditions. Exhibit 35-5 shows an example of variable speed limit signs used for speed harmonization in the Netherlands.

Source: Federal Highway Administration, ATM Scan, Jessie Yung.

Exhibit 35-5
Variable Speed Limit Signs, Rotterdam, Netherlands

TRAFFIC SIGNAL CONTROL

Signal timing optimization is the single most cost-effective action that can be taken to improve a roadway corridor's capacity and performance (*4*). Signal timing is equal in importance to the number of lanes in determining the capacity and performance of an urban street.

Traffic signal timing optimization and coordination minimizes the stops, delays, and queues for vehicles at individual and multiple signalized intersections.

Traffic signal preemption or priority provides special timing for certain classes of vehicles such as buses, light rail vehicles, emergency response vehicles,

and railroad trains. Preemption interrupts the regular signal operation. Priority either extends or advances the time when a priority vehicle obtains the green phase, but generally operates within the constraints of the regular signal operating scheme.

Traffic-responsive operation and adaptive control provide for different levels of automation in the adjustment of signal timing due to variations in demand. Traffic-responsive operation selects from a prepared set of timing plans based on the observed level of traffic in the system. Adaptive traffic signal control involves advanced detection of traffic, prediction of its arrival at the downstream signal, and adjustment of the downstream signal operation based on that prediction.

SPECIALIZED APPLICATIONS OF ATM STRATEGIES

ATM strategies are often applied to the day-to-day operation of a facility. Incident management and work zone management are examples of applications of one or more ATM strategies to address specific facility conditions. Employer-based demand management is an example of private-sector applications for which traveler information systems may be an important component.

Incident Management

Traffic incident management is "the coordinated, preplanned use of technology, processes, and procedures to reduce the duration and impact of incidents, and to improve the safety of motorists, crash victims, and incident responders" (*5*). An incident is "any non-recurring event . . . that causes a reduction in roadway capacity or an abnormal increase in traffic demand that disrupts the normal operation of the transportation system" (*5*). Such events include traffic crashes, disabled vehicles, spilled cargo, severe weather, and special events such as sporting events and concerts. ATM strategies may be included as part of an overall incident management plan to improve facility operations during and after incidents.

Work Zone Management

Work zone management has the objective of safely moving traffic through the working area with as little delay as possible consistent with the safety of the workers, the safety of the traveling public, and the requirements of the work being performed. A transportation management plan (TMP) is a collection of administrative, procedural, and operational strategies used to manage and mitigate the impacts of a work zone project. The TMP may have three components: a temporary traffic control plan, a transportation operations plan, and a public information plan. The temporary traffic control plan describes control strategies, traffic control devices, and project coordination. The transportation operations plan identifies the demand management, corridor management, work zone safety management, and traffic and incident management and enforcement strategies. The public information plan describes public awareness and motorist information strategies (*5*). ATM strategies can be important components of a TMP.

Employer-Based Demand Management

Employer-based demand management consists of cooperative actions taken by employers to reduce the impacts of recurring or nonrecurring traffic congestion on employee productivity. For example, a large employer may implement work-at-home or stay-at-home days in response to announced snow days; "spare the air" days; or traffic alerts regarding major construction projects, major incidents, and major highway facility closures. Another company may contract for or directly provide regular shuttle van service to and from transit stations. Flexible or staggered work hours may be implemented to enable employees to avoid peak commute hours. Ridesharing services and incentives may be implemented by the employer to facilitate employee ridesharing.

Employers may also use components of a traveler information system to determine appropriate responses to changing traffic conditions. Employees can use traveler information systems in their daily commuting choices.

3. METAMEASURES OF EFFECTIVENESS

INTRODUCTION

This section describes the need for meta-MOEs for evaluating ATM strategies and provides some candidates for consideration. Meta-MOEs are combinations of traditional HCM MOEs that have been computed over a range of demand and capacity conditions expected to occur in the real world.

NEED FOR META-MOEs

The analysis methodologies described elsewhere in the HCM are designed to produce a single set of performance results for a given set of input demands and computed capacities for a facility. Volume 1 provides discussions of the performance measures produced by the HCM for each system element in Chapter 4, Traffic Flow and Capacity Concepts; Chapter 5, Quality and Level of Service Concepts; and Chapter 7, Interpreting HCM and Alternative Tool Results. These HCM MOEs are, in essence, single point estimates of facility performance.

In addition, the HCM methodologies described elsewhere in this manual are often specifically oriented to ideal or near-ideal conditions, when weather, incidents, and other factors do not adversely affect capacity. HCM methodologies can be adapted to account for adverse effects on capacity, but their default condition is to exclude these effects.

ATM strategies are designed to improve the performance of the facility over a range of real-world demand and capacity conditions, not just for a single forecast condition.

ATM strategies, however, are designed to improve the performance of a facility over a range of real-world demand and capacity conditions, not just for a single forecast condition. Thus, the standard HCM performance measures and methodologies exclude the majority of the benefits of the dynamic and continuous monitoring and control of the transportation system, which is the objective of ATM.

The evaluation of ATM strategies requires performance measures that recognize the impact of less-than-ideal conditions on facility capacity.

A methodology is needed for computing traditional HCM MOEs (such as density, delay, speed, volume-to-capacity ratio, and queues) over a range of likely demand and capacity conditions and to combine them into one or more meta-MOEs that better characterize system performance under real-world conditions.

CANDIDATE META-MOEs

The evaluation of ATM performance requires MOEs that quantify the impacts of varying demands and capacities on performance. One way to achieve this is to develop methods for computing traditional HCM MOEs for varying combinations of demand and capacity conditions and to combine the results into various meta-MOEs for describing system performance with varying ATM strategies.

Various meta-MOEs may be considered by the analyst. These include

- Measures of central tendency, such as the mean, mode, or median of the HCM results;

- Measures of variation, such as the standard deviation or the variance of the HCM results;
- Measures of extreme results, such as the worst HCM results at the 85th, 90th, 95th, or 99th percentile;
- Measures of probability of failure and duration of failure, such as the probability of exceeding a target demand-to-capacity ratio for a given length of time, the probability of exceeding a target level of service, or the probability of exceeding some other agency-determined threshold MOE; and
- Measures of production, such as throughput, vehicle miles traveled, vehicles served, person miles traveled, or persons served.

For example, the analyst may choose to report for a traffic signal the mean delay, the standard deviation of delay, the 95th percentile delay, the probability of exceeding LOS E, the total number of vehicles served (throughput), or some combination of these measures. Each of these measures would be computed by using HCM methods for varying combinations of demand and capacity; the results would then be combined into meta-MOEs for the signal.

At present the interpretation and determination of what constitutes acceptable or unacceptable meta-performance is an open question that requires further research.

INDICES OF PERFORMANCE

While using many different MOEs can give a more complete picture of system performance, sometimes the data become too massive to comprehend, thus hindering rather than assisting the decision-making process. In such a case the analyst may find it desirable to combine one or more of the meta-MOEs of ATM performance into a single index. Performance indices are also useful when the analyst desires to optimize multiple dimensions of system performance. For example, signal timing optimization usually involves optimizing a weighted combination of stops and delays.

The formula in Equation 35-1 provides one example of many potentially useful methods for combining meta-MOEs into a meaningful index of performance. It applies an analyst-defined percentage weighting W to the average system performance and one minus that percentage to the 95th percentile system performance to yield an assessment of the robustness of the system. Other combinations that may be more useful to the specific needs of the analysis are also possible.

$$\text{Robustness Index} = W \times (\text{Average MOE}) + (1 - W) \times (95\%\ \text{MOE})$$

Equation 35-1

where

Robustness Index = example composite index of system robustness,

W = relative weight (between 0 and 1), and

MOE = HCM MOE.

4. GENERAL EFFECTS

INTRODUCTION

This section presents basic information on what are considered to be the likely effects of specific ATM strategies on the demand, capacity, and performance of a roadway facility. The reader should recognize that there are currently many gaps in this basic information and that much of this discussion is based on a sparse set of research results.

ROADWAY METERING

Demand Effects

Roadway metering shifts some of the demand for the facility to other routes, other modes, and other times of day. Some of the demand remains, simply waiting for its turn to enter the facility. The demand effects are specific to the situation and the alternatives available.

Capacity Effects

Freeway on-ramp meters have been found to increase freeway mainline bottleneck capacity by 3% to 5% (*6*, *7*). This effect is achieved by smoothing the microsurges of traffic from the on-ramp impacting the freeway and thereby delaying breakdown conditions at the bottleneck (*8*).

Greater increases have been observed in mainline vehicle throughput measured at various points upstream of a bottleneck.

Performance Effects

The primary performance effect of roadway metering is to delay or prevent the onset of mainline traffic congestion or breakdown. Average speeds of traffic within the metered facility can be significantly improved. The trade-off is increased delays for vehicles at the meters. A systemwide assessment is required to determine net system benefits.

Ramp meter evaluation studies (*9*) found that when freeway on-ramp meters were turned off

- Freeway volumes dropped 9%,
- Peak period freeway throughput declined 14%,
- Freeway travel times increased 22%,
- Freeway speeds dropped 7%, and
- Freeway crashes increased 26%.

Installing ramp meters would be expected to have the opposite results of those cited above (i.e., increased volumes, increased throughput, increased speeds, and fewer crashes). The performance benefits of roadway metering will vary with the specific conditions of each installation.

Estimation Methods

The HCM methodologies described elsewhere in this manual can estimate the performance effects of roadway metering. These methods, however, do not currently recognize the bottleneck capacity increases that are provided by freeway on-ramp metering.

Microsimulation models that have been properly calibrated to field conditions can be used to model the supply-side effects of ramp metering, mainline metering, and peak period ramp closures on freeway capacity and performance.

Demand models employing traffic assignment methods sensitive to metering (such as dynamic traffic assignment) are often required to estimate demand-side effects.

Capacity of Metered Freeway On-Ramps

A single-lane metered on-ramp that allows one vehicle per green can serve up to 900 veh/h. If two vehicles are allowed per green, then a single-lane metered on-ramp can serve from 1,100 to 1,200 veh/h (*10*).

A two-lane metered ramp provides a capacity of 1,600 to 1,700 entry vehicles per hour across the two lanes of the on-ramp (*10*).

These values are approximate. Actual capacity is determined by the maximum feasible metering rate, driver aggressiveness, and the ability of the freeway to absorb the ramp volume. While higher metering rates may be theoretically possible, practical constraints (such as driver compliance and reaction times) limit the maximum and minimum metering rates that may be employed.

CONGESTION PRICING

Demand Effects

Congestion pricing shifts some of the demand to other lanes, other routes, other modes, and other times of day. Some of the demand remains, and drivers will simply pay the toll. The demand effects are specific to the pricing policy, the travelers' value of time, and the alternatives available.

If the pricing policy is to maintain demand on the facility within a target range, then the demand for the facility is the known value (within the target range), and the unknown value is the price.

If the pricing policy is to maintain a minimum speed on the facility, then the equivalent maximum operating volume range is on the order of 1,600 to 1,700 veh/h/ln (*11*). These values appear to be appropriate for sustained minimum average operating speeds of 40 to 45 mi/h for a single HOT lane. Lower flow values may be necessary to achieve higher average sustainable minimum operating speeds. Higher flow values may be achievable for multilane HOT lane facilities.

Capacity Effects

Congestion pricing, by spreading out the peaking of facility demand, can enable the facility to move more vehicles over the course of a peak period.

Performance Effects

Congestion pricing can result in significant reductions in delay for the priced facility. A study of the CA-91 express lanes (*12*) found the following:

- Overall traffic volumes on CA-91 increased by 15% in the first 18 months after express lanes were added to the facility.
- Peak-direction travel times for express lane users were reduced from 70 min before the express lanes opened to 12 min after the lanes opened. Non–express lane users also experienced a significant reduction in peak-direction travel times, from 70 min to 30 min. However, increasing demand over the following 18 months gradually eroded much of that savings for the non–express lane users.
- The express lanes did not cause a significant change in vehicle occupancies.

Estimation Methods

The demands for a priced facility can often be reasonably estimated from the pricing policy if the pricing policy sets a minimum operating speed threshold.

A demand model and the value of time are required for predicting systemwide demand effects, for evaluating the demand effects of specific pricing schedules, and for estimating revenues.

At present the HCM does not provide methodologies for evaluating many of the specific geometric configurations currently being used to implement HOT lanes. Capacity and operation analysis methods are lacking for single-lane facilities where faster vehicles are unable to pass a slower vehicle in the lane. The entry and exit points for barrier-separated facilities are also not explicitly covered by HCM methodologies, although the methodologies may be adaptable to those conditions.

Microsimulation models, properly calibrated to field conditions, may be used to evaluate the operation of congestion-priced facilities.

TRAVELER INFORMATION SYSTEMS

Demand Effects

Traveler information systems shift some of the demand to other routes, other modes, and other times of day. Some of the demand will remain. The demand effects are specific to the situation and the alternatives available.

Capacity Effects

Traveler information systems, by redirecting demand, can postpone or avoid the onset of traffic congestion, thus yielding the throughput benefits typical of such conditions.

Performance Effects

Reductions in demand due to redirected traffic and the postponement of traffic breakdowns can result in net performance improvements. Work zone management programs in Texas and Washington, D.C., employing traveler information systems as part of an overall ATM strategy to improve traffic operations within work zones have achieved demand diversions of between 10% and 50% (*13*).

Estimation Methods

The HCM does not provide methodologies for directly assessing the performance effects of traveler information systems. However, if an estimate of the changed demand levels can be obtained, then the HCM methodologies can be applied to estimate system performance.

Some microscopic and mesoscopic simulation models provide route choice algorithm parameters that can be adjusted to account for different levels of traveler information penetration and compliance in the vehicle fleet.

Demand-forecasting tools have not been typically used to predict the demand effects of traveler information systems, but they may be adaptable for that purpose. The analyst should verify how the demand-modeling software treats traveler information within its route-choice process.

MANAGED LANES

Demand Effects

Managed lanes change the nature and quantity of demand for a facility. Capacity increases due to the addition of managed lanes tend to draw more demand to the facility. Managed lanes can cause modal and temporal shifts in demand for the facility by making certain modes of travel subject to less delay than others for certain times of the day.

Capacity Effects

The addition of new managed lanes to a facility generally increases the facility's overall capacity.

For managed lanes that are barrier separated from the rest of the facility, weaving and merging at the entry and exit points may be a significant traffic operations issue. The weaving capacity of the entry and exit points may control the overall facility capacity (*14*). The capacity would be affected by mainline configuration, access design, and traffic patterns.

For reversible lanes, significant capacity and performance benefits may be lost when the lanes must be closed in their entirety so that the flow direction can be reversed.

Performance Effects

The addition of new managed lanes to a facility generally improves facility performance for all users. Vehicles eligible to use the HOV lanes will experience significant reductions in delay. Single-occupant vehicles will also experience

reduced delays because of the additional gaps in traffic opened up when HOVs move from the mixed-flow lanes to the HOV lanes.

A Federal Highway Administration inventory of HOV facilities in the United States (*15*) found that HOV lane users experienced travel time savings of between a few seconds per mile to 6 min/mi or more, depending on the extent and severity of congestion on the adjacent mixed-flow lanes.

Estimation Methods

The HCM methodologies described elsewhere in this manual are not validated or calibrated for the special conditions posed by managed lanes or shoulder lanes; however, it may be possible to adapt the HCM methods with the proper choice of parameters.

Microsimulation tools that are properly calibrated and validated for existing field conditions can provide performance information for most managed lane configurations. Special calibration of these models may be required to model shoulder lanes adequately.

Maximum Target Flow Rates for HOV and Bus Lanes

HOV lanes start to experience a noticeable degradation of performance (speeds dropping to 45 mi/h or less) at flows of 1,200 to 1,500 veh/h/ln (*16*). The following general maximum operating thresholds for different types of HOV facilities are based on national experience (*17*):

- Separate right-of-way, bus only: 800 to 1,000 veh/h/ln
- Separate right-of-way, HOV: 1,500 to 1,800 veh/h/ln
- Freeway, exclusive two directional: 1,200 to 1,500 veh/h/ln
- Freeway, exclusive reversible: 1,500 to 1,800 veh/h/ln
- Freeway, concurrent flow: 1,200 to 1,500 veh/h/ln
- Freeway contraflow, bus only: 600 to 800 veh/h/ln
- Freeway contraflow, HOV: 1,200 to 1,500 veh/h/ln
- HOV bypass lanes: 300 to 500 veh/h/ln

Note that the above maximum operating thresholds are not capacities, but rather values above which an undesirable degradation in lane speeds is likely to occur. These values generally apply to single-lane operation. Multiple lanes may achieve higher operating thresholds.

The *Transit Capacity and Quality of Service Manual* (*18*) points out that the capacity of exclusive bus lanes on freeways is dictated either by the capacity of any off-line bus stops along the bus lane section or by the bus stops located after the end of the bus lane. Thus the capacity of a bus lane on a freeway is generally meaningless.

TRAFFIC SIGNAL CONTROL

Demand Effects

Traffic signal control, by controlling capacity and delay, can draw more demand to the facility or can shift some of the demand to other routes, other modes, and other times of day. The demand effects are specific to the conditions and the alternatives available.

Capacity Effects

Traffic signal control directly affects capacity through the formula shown in Equation 35-2.

$$c = (g / C) \times s$$

Equation 35-2

where

c = capacity (veh/h),

g/C = effective green time per traffic signal cycle length, and

s = saturation flow rate (veh/h).

ATM strategies that modify the heavy-vehicle mix can influence the saturation flow rate, and those strategies that affect peaking can influence the peak 15-min volume-to-capacity ratio. Otherwise, signal control affects capacity primarily through the g/C ratio.

Performance Effects

The effects of advanced signal timing applications vary according to the quality of the signal timing plans in place prior to implementation. The percentage change can be small if the original plan was of high quality and frequently maintained and updated.

On average, improvements to signal timing plans have been found to reduce average peak period facility travel times by 8% to 25% and to reduce delay in the 15% to 40% range (*19*).

Estimation Methods

The HCM methodologies described in Volume 3 can be used to evaluate the capacity and performance effects of the optimization and coordination of fixed-time and traffic-actuated signal systems. These methods, however, are not suitable for evaluating signal preemption, signal priority, or traffic-adaptive and traffic-responsive control strategies.

Most commonly available microsimulation tools are appropriate for evaluating signal control strategies. Their ability to model advanced control strategies (traffic-responsive and -adaptive controls) varies according to the sophistication of the signal controller emulator built into the microsimulation tool.

SPEED HARMONIZATION

Demand Effects

The available literature on speed harmonization does not provide information on its demand effects. If speed harmonization does not significantly change average travel speeds for the facility, then it would not be expected to affect demand significantly. Improved operations and reliability associated with speed harmonization might draw demand to the facility.

Capacity Effects

Speed harmonization is designed to reduce the frequency of incidents caused by sudden decelerations in the traffic stream and to postpone the onset of congestion. These effects will in turn influence the facility capacity.

Some early studies found that speed harmonization could increase the total capacity of a freeway by 10%, but other studies have found no effects. More recent studies in Germany suggest that the primary capacity impact of speed harmonization is on the variation of capacity. Capacity variance may be reduced 50% while average capacity is increased on the order of 3% (*20*). Speed harmonization on the Netherlands' Motorway Control System was found to increase vehicle throughput by 3% to 5% (*21*).

Performance Effects

Studies to date suggest clear benefits in terms of collision reductions, which translate into better reliability.

The literature is less clear on the performance effects of speed harmonization. Speed harmonization often results in lower average speeds, which are counterbalanced to some extent by improved reliability in travel times. These counterbalancing effects can result in net positive or negative travel time benefits, depending on circumstances. For example, a freeway management plan that included speed harmonization on the M25 controlled motorway in the United Kingdom was found to result in a 10% reduction in injury collisions, no net change in travel times, and a 9% reduction in time the facility was operating in flow breakdown conditions (speeds under 25 mi/h) (*22*).

Estimation Methods

The HCM methodologies described elsewhere in this manual do not recognize the potential capacity effects of postponing breakdown or the reliability effects of reduced incident frequency.

Most commonly available microsimulation models will show the performance changes of reducing speed variance and shocks in the traffic stream, but using them to model the reliability and delay effects of reduced incidents is more difficult. A methodology is required to estimate the reduced probability of incidents for a given speed harmonization policy.

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